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PROCEEDINGS
OF
THE SECOND WORLD CONFERENCE
ON
EARTHQUAKE ENGINEERING

Tokyo and Kyoto, Japan

July 11—18, 1960

Volume III

Conference Organized by the

SCIENCE COUNCIL OF JAPAN

in cooperation with the

JAPAN SOCIETY OF CIVIL ENGINEERS
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SEISMOLOGICAL SOCIETY OF JAPAN

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SEISMICITY AND EARTHQUAKE GROUND MOTION

(SESSION III)

The questions not answered in a written form by the author do not appear on the pages of discussion.

Seismicity of Japan

Takuzo Hirono*

Japan belongs to the Circum-Pacific Seismic Zone and its seismic activity is more vigorous than any other place of the same zone. Japan experiences earthquakes which are perceptible somewhere on Japan about 1497 times in average year from 1923 to 1959 but they very considerably from year to year (See Fig.1) and become numerous after any big earthquake or by occasional earthquake swarms. The maximum number for the last 37 years occurred in 1930 amounting to 5770 while the minimum in 1958 was 850. The former was caused by the earthquake swarms which occurred near Hakone..[1].

The geographical distribution of the number of earthquakes which were felt by persons is shown in Fig.2 which is made on the basis of the data of the last 38 years. It indicates that the most frequently felt region is that along the Pacific coast of east and north Japan, while other regions are not so frequent, and the most quiet region is north Hokkaido. Fig.3 is a similar chart made by the late Prof. Omori [2] showing the total number for 6 years from 1885 to 1890 and it shows also that the Pacific coast of east Japan is most frequent. This fact indicates that the trend of the recent seismic activity has not changed very much since that time.

In Fig.2, the largest number of perceptible earthquakes was observed at Tsukuba to the east and Wakayama to the west both of which amount to 137 times per year as the mean value of from 1921 to 1958. [3] This value means that about 18% of the total felt earthquakes in Japan are occurring in these two localities. The former is situated within the most frequently felt region of the Pacific coast, but the latter is isolated outside of that region. The earthquakes of Tsukuba occur mostly at depths of between 30 to 60 Km, while those of Wakayama occur, shallower than 30 Km. Though these two places are habitually occurring regions, sometimes there are temporarily local swarms of earthquakes especially at volcanic regions.

That the most frequently felt region lies along the Pacific coast of north and east Japan suggests that there is a zone of seismic activity off these coasts. Fig.4 which shows the distribution of the epicenters of earthquakes during 1926-1956 assures this suggestion. All these earthquakes have the magnitudes such that the diameters of their perceptible areas are larger than 200 Km, that is, about 4.0 in M scale (the magnitude scale of Gutenberg and Richter) and let us call them A group. Fig. 4 shows that the seismic activity concentrates off the coast of the Kanto District. It extends northwards under the Pacific Ocean along the coast of east and north Japan, while in west Japan it disperses evenly over the land and less active under the sea far off that coast. [4].

The number of earthquakes mentioned above is about 142 in the average year as seen in Fig.5 and it is about 10% of the average number of all perceptible earthquakes. Let us call the latter group as F: it involves A in it. Secular variations of these two groups are very different from each other. While F group has been diminishing recently, A group has been increasing. The yearly number of earthquakes with a perceptible area

* Japan Meteorological Agency

of larger than 600 Km in diameter is also shown in Fig.5 as B group. Their magnitudes are larger than 5 ¾ in M scale. They occur about 23 times in the average year, and, hence, it is 16% of A group.

The earthquake which could cause casualties has a magnitude larger than 6. The distribution of such dangerous shocks which occurred during recent 31 years is shown in Fig.6. It also shows that the largest activity is under the Pacific Ocean similar to Fig.4. But as they originated far off the coast, the majority of these shocks ended in harmlessness. Even the earthquakes of the order of 7 in M scale which could kill few thousand persons if it occurs under a crouded city remained in causing a weak tsunami at the worst.

The data of 31 years above mentioned shows that the earthquakes of magnitude larger than 6 occur 16 times in average year, of which 4 are deeper and 12 shallower than 60 Km, and among shallower ones, 10 occur under the sea and 2 on land of which only one causing damage. Those larger than 7.0 in M scale occur twice in average year of which one is deep and once every 3 years on land.

It is true that earthquakes of the order of 7 in M scale occurred occasionally on land, but the earthquakes of the order of 8 did not happen on land during 31 years now under consideration. They occurred under the Pacific Ocean, and they were only 4 times. The last earthquake of the order of 8 which occurred on land is the Mino-Owari Earthquake of 1891 (M: 8.4) which is famous for a fault traced to a distance of about 100 Km. It demolished or badly damaged 142,177 hauses and killed 7273 people.

Some of the earthquakes of the order of 8 which occurred under the ocean brought large tsunamis destroying harbours and fishing villages. As a matter of fact, when the submarine origin is under the shallow sea near the land the direct damage of shock is comparatively more severe than the damage due to tsunami as in the case of the Nankaido Earthquake of 1946, while when the origin locates far off the coast under the deep sea bottom, it causes big tsunami devastating the coasts nearest to the origin. This devastation is far greater than the direct damage of shock as in the case of the Sanriku Earthquake of 1933.

As stated above the sea bottom off the coast of the Pacific Ocean of north and east Japan is most vigorous in seismic activity but big earthquakes of the order of 8 in M scale occur also under the Pacific Ocean off the coast of west Japan, and though Fig.6 indicates their epicenters as points, the regions of crustal deformation due to them are thought to occupy considerably large areas. Fig.7 which was made by Mr. Musha[5] shows the epicentral regions of destructive earthquakes during the past two thousand years. It shows the largest areas correspond to the earthquakes of the order of 8 in M scale. When large epicentral areas are also taken as active places, the zone of seismic activity of the Pacific coast is thought to be continuous throughout north, east and west Japan and this is called the Outer Seismic Zone. On the other hand, several earthquakes of the order of 7 in M scale have occurred along the coast of the Japan Sea forming another seismic zone which is called the Inner Seismic Zone. Besides these, several minor ones seem to exist

and earthquakes occurring there do not always cause less casualties than those of the two major zones.

Now, it must be noted that in Fig.4 and 6, only the epicenters shallower than 60 Km in depth are shown. As far as seismicity is concerned it would be inadequate to omit deep earthquakes, but there is no record of damage by any eathquake which originated at a depth deeper than 80 Km. Therefore it is not so significant fron the point of view of earthquake engineering that we confine ourselves to show the geographical distribution of deep earthquakes during 1926-1956 (Fig.8). Earthquakes deeper than 200 Km form themselves into seismic zones other than those for shallow earthquakes.

The earthquakes by which damage was caused in Japan since 1900 have amounted to 70 in number[6], among these those bringing the death roll of more than 10 were 24, that is, Japan has about one of these per year. The earthquake which caused greatest number of casualties in Japan was the Kanto Earthquakes of 1923 by which 576,000 houses were destroyed or burnt down and 143,000 persons were killed or missing. The next is the Fukui Earthquake of 1948 in which 39,000 houses were burnt down and 3,579 persons were killed.

One of the most remarkable features of the epicentral region of large earthquakes is its habit of repetition of earthquake's occurrence. For instance, in the Pacific Ocean off west Japan large earthquakes have occurred 7 times or more in almost the same place in the past two thousand years. This is a very important fact as it suggests that the place where people have experienced severe earthquakes will probably be visited some day by another earthquake of a similar intensity. Therefore, the intensity distribution of large earthquakes is very interesting. Fig.9 is an example showing the intensity distribution in the case of the Nankaido Earthquake of 1946.

The scale of earthquake intensity which is now used in Japan consists of 7 grades excluding no feeling (See Table 1). Though determined by the feeling of human body, observed intensity was found to have a considerably close relation with the maximum acceleration of ground motion and it is given by Prof. Kawasumi[7] as

$$a_n = 0.8\,(10^{\frac{n-1}{2}} - 10^{\frac{n}{2}})\ \text{gal} \qquad \text{for } 1 \leq n < 7$$
$$a_n < 0.8\ \text{gal},$$
$$a_7 > 250\ \text{gal}.$$

where a_n is the acceleration corrasponding to the intensity n, and the 7^{th} grade of the intensity scale was created after the Fukui Earthquake of 1948.

Damage to houses begins to appear when the intensity attains to 5 grade and houses fall down when it becomes 6 grade. Fig.10 shows the distribution of the number of occurrences of intensities larger than 4 for the last 38 years. It shows that earthquakes with 5,6 and 7 intensity grades occur most frequently in the Kanto District which is Tokyo and its surrounding area, and actually the place where earthquakes of these intensities occur more frequently than any other places is an

isolated point, Kofu. The observed number at Kofu is 13 for the same period, but all of them occurred in 1923, the year of the great Kanto Earthquake.

Besides the 108 weather stations of J.M.A. (Japan Meteorological Agency) there are about 1,100 weather sub-stations where seismic intensity is observed without instruments, and all of these stations send in reports on the observations they have made to the J.M.A. This data, especially data regarding the maximum earthquake intensity in a certain place, is made available to construction engineers before they begin to erect constructions of any kind, i.e. buildings, embankments, bridges etc.

As it is impossible for one station to determine the exact origin of earthquake the above mentioned 108 stations, each being equipped with seismographs of one and 100 in magnification, form a network and work together in order to ascertain an earthquake's origin, and the location and the intensity distribution of the earthquake felt by persons are generally announced officially about 50 minutes after the occurrence of the earthquake and when there is a fear of tsunami by a submarine earthquake, a tsunami warning is to be issued from some of 8 tsunami warning centers within 20 minutes after the earthquake's occurrence. Then, of course, the network of communication needs to work at its highest speed so that the warning is in time for the arrival of tsunami.

Casualties by earthquakes are tending to increase because city populations are growing larger and larger and large industrial constructions such as factories etc. are becoming more and more. In order to prevent such casualties not only earthquake proof engineering but also earthquake prediction is important. Consequently, seismologists in Japan who have the facilities to do so are concentrating their efforts on this problem of earthquake prediction. This has been one of the major problem facing seismologists in Japan since the days of Prof. Omori and Imamura.

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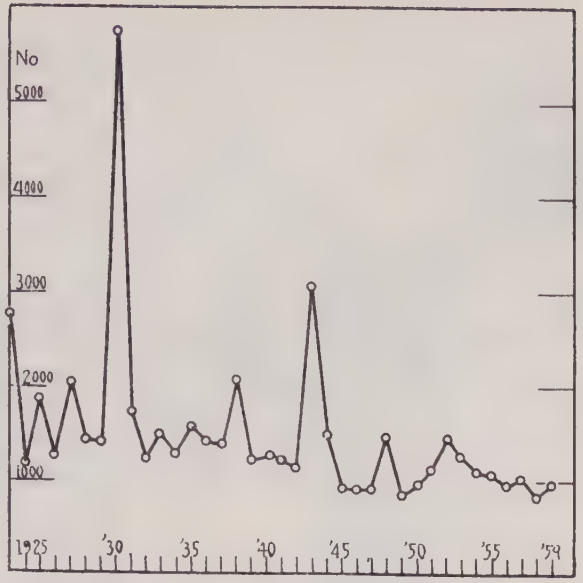

Fig. 1 The secular change of the number of earthquakes felt in Japan in each year.

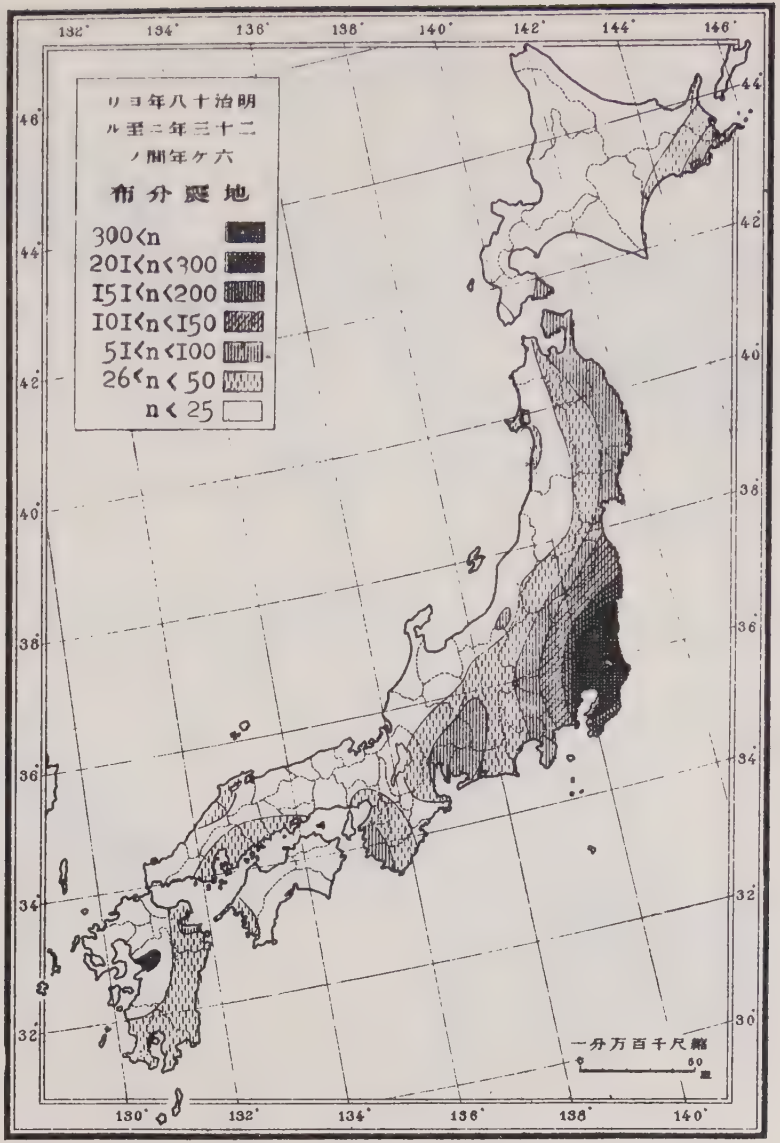

Fig. 3 The distribution of the numbrt of felt earthquakes during 6 years from 1885 to 1890 (Taken from Prof. Omori's Jishingaku-kowa)

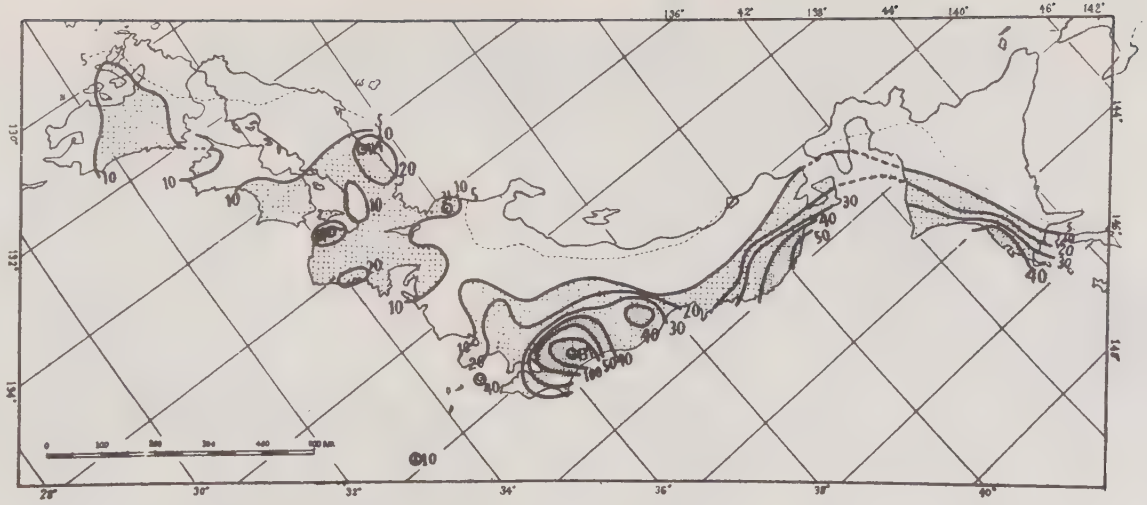

Fig. 2
The distribution of the number of felt earthquakes in the avarage year from 1921 to 1958.

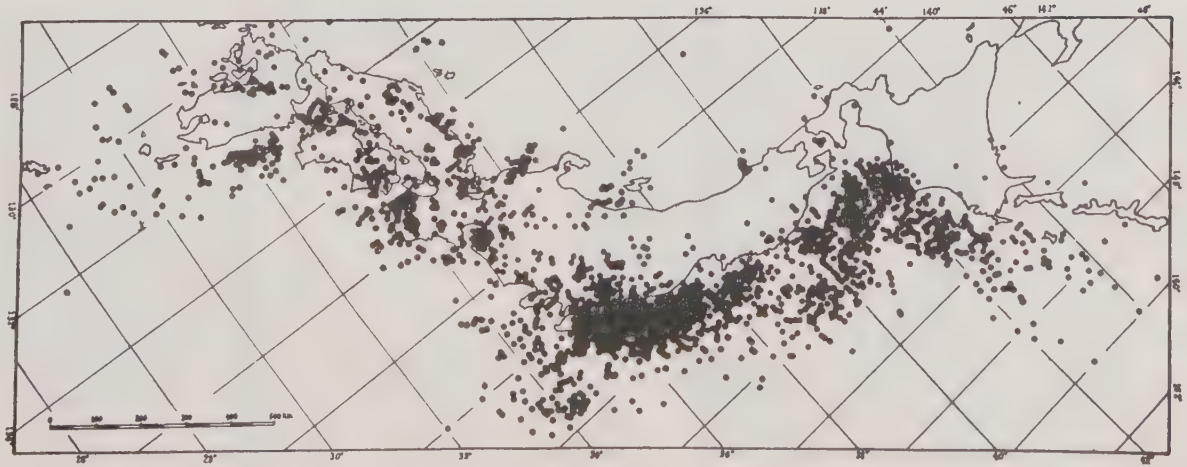

Fig. 4
The distribution of the centers of earthquakes larger than that with the felt area of the diameter of 200 km (1926-1956).

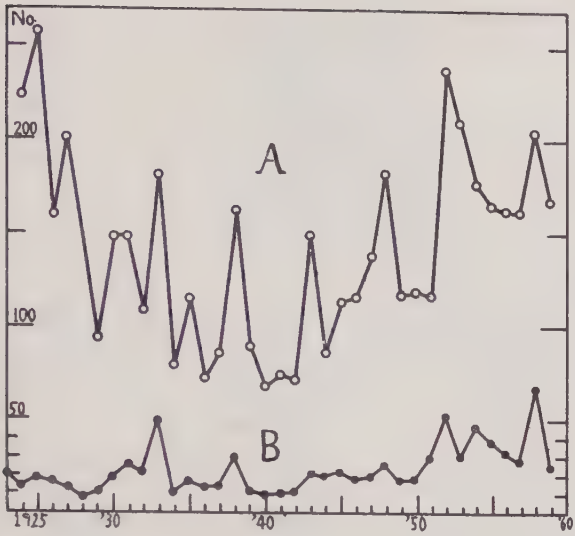

Fig. 5

The secular changes of the number of earthquakes which occurred in Japan in each year: A group—the diameter of the felt area is larger than 200 km. B group—the diameter larger than 600 km.

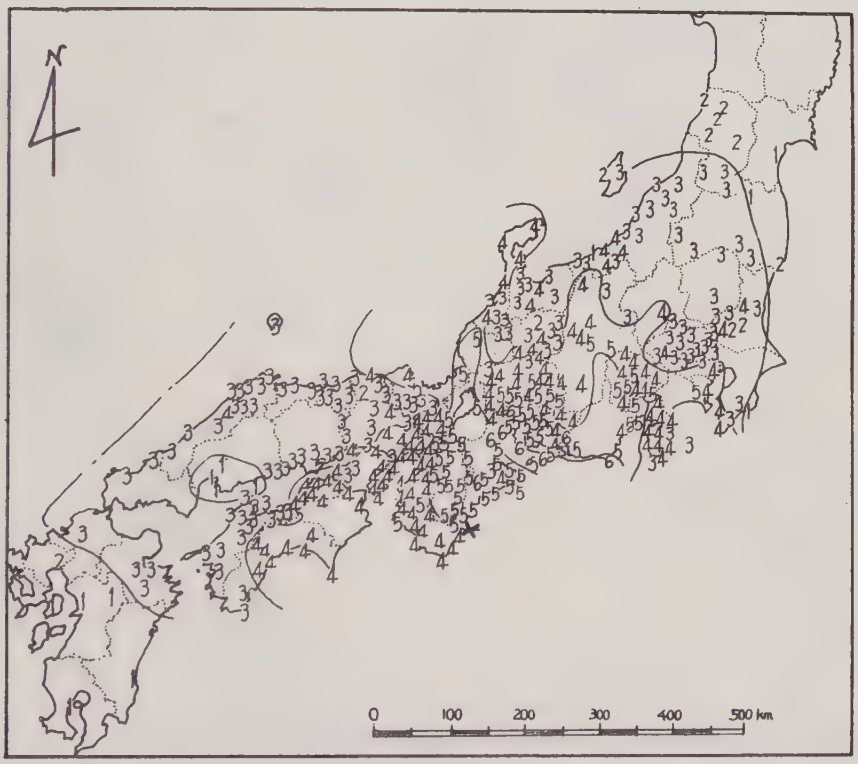

Fig. 9 The intencity distribution due to the Tonankai Earthquake of 1944.

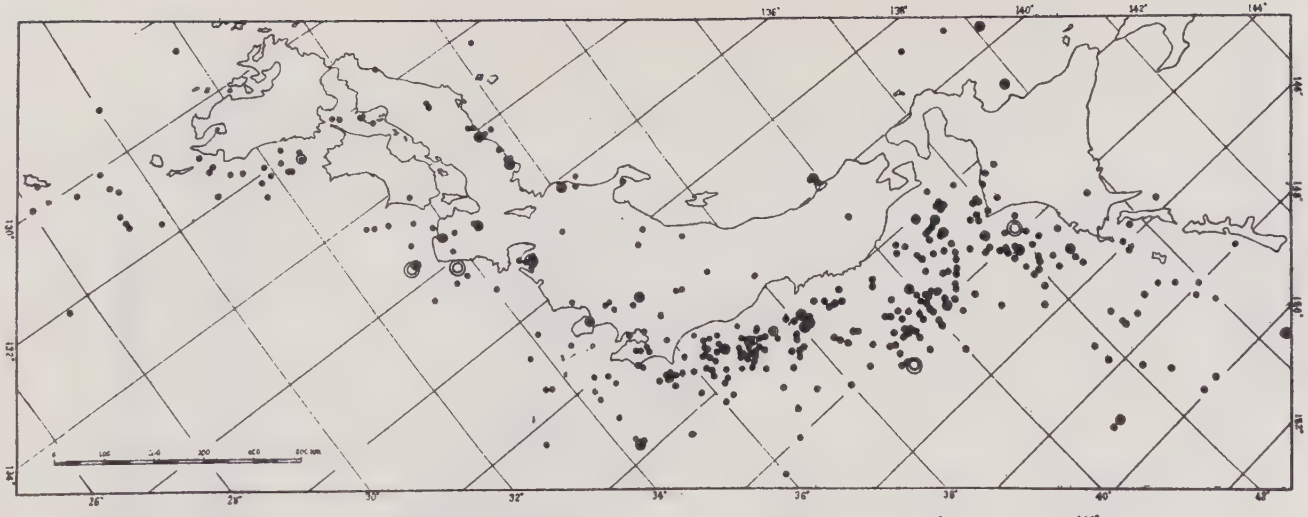

Fig. 6

The distribution of the centers of earthquakes larger than that of magnitude 6 which occurred during 1926-1956.

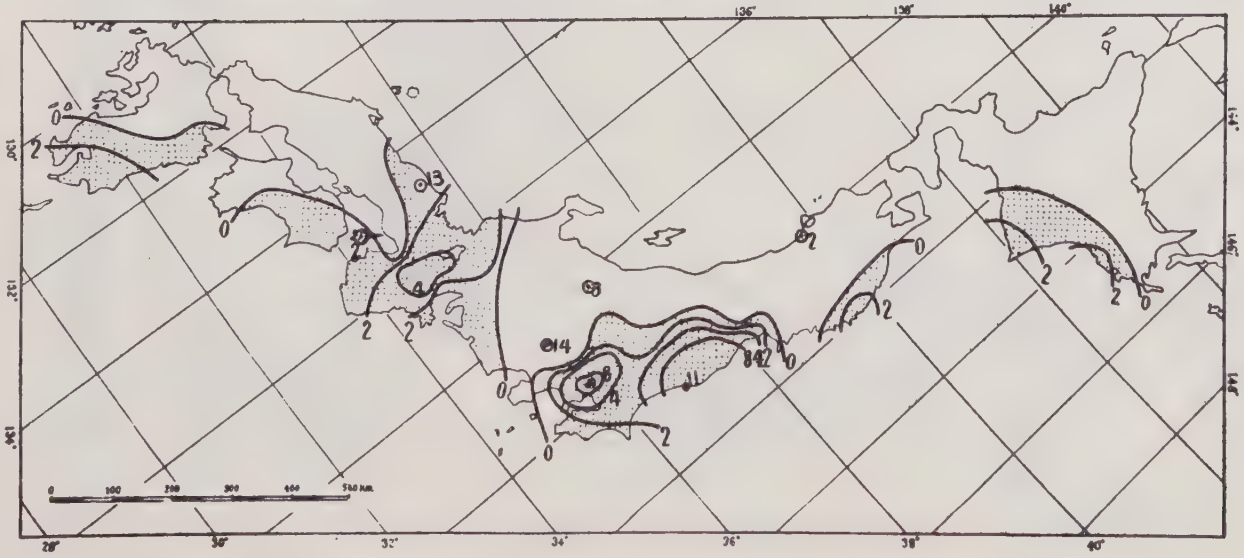

Fig. 10

The distribution of the observed number of intensities larger than the 4th grade during the years from 1921 to 1958.

Fig. 7 The distribution of the epicentral areas of the destructive earthquakes which occurred during Christian era.
The black area occurred before 1890. Numerals mean the number of occurrences at the same place.

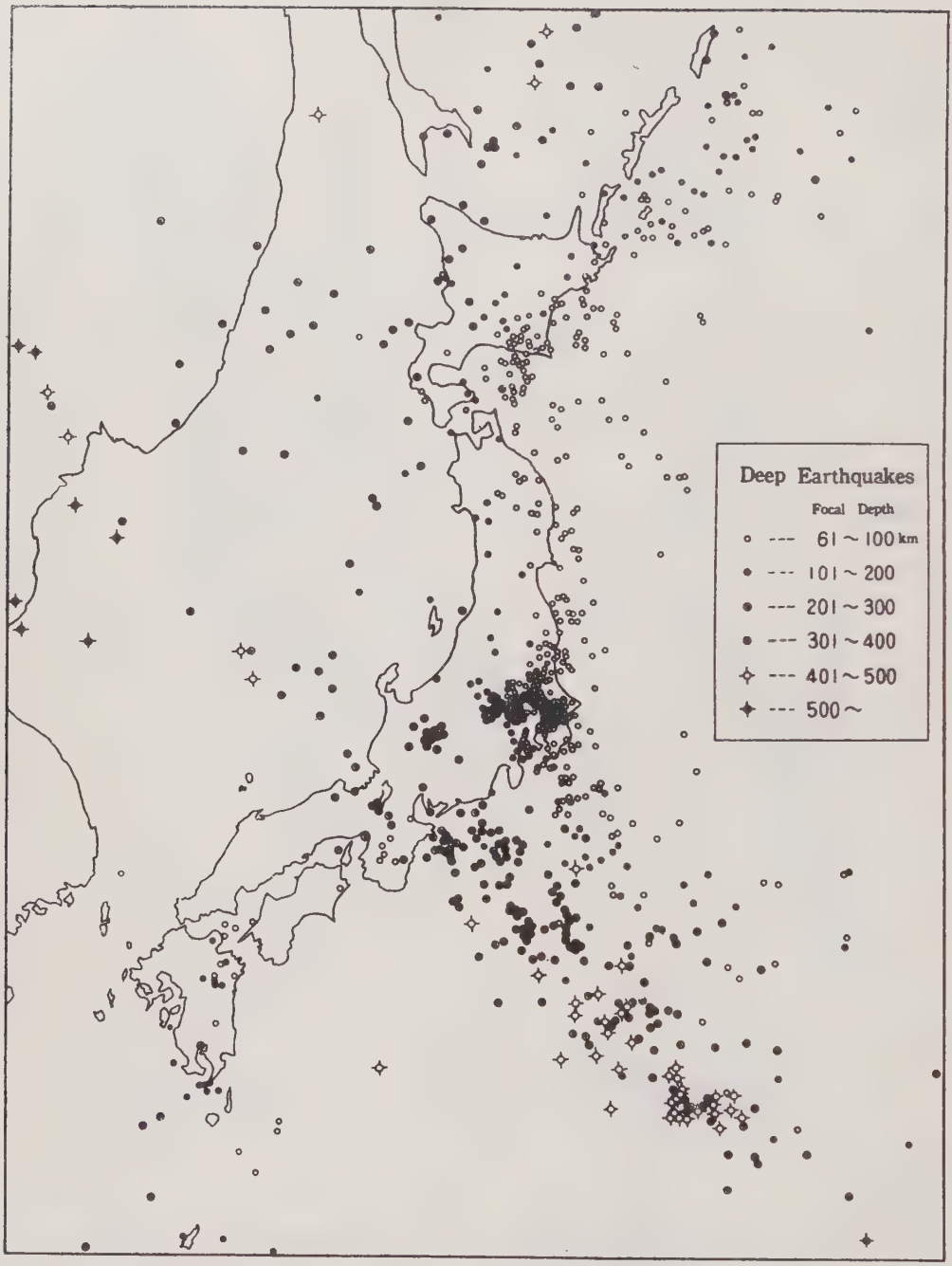

Fig. 8 The distribution of the earthquakes whose focal depths are deeper than 60 km (1926 - 1956).

Table 1 Scales of seismic intensity

The intensity of the shock is estimated according to the scales 0 - VII as follows :-

	No Feeling	Slight	Weak	Rather Strong	Very Strong	Very Disastrous	Very Disastrous
Scale	0	I	II	III	IV	V	VI	VII

0; No Feeling: Shocks too weak to cause human feelings, registered only by a seismograph.
I; Slight: Extremely feeble shocks only felt by persons at rest or by those who are observant to an earthquake.
II; Weak: Shocks felt by most persons, slight shaking of doors and Japanese latticed sliding doors (Shoji).
III; Rather Strong: Slight shaking of houses and bildings, rattings of doors and Japanese latticed sliding doors (Shoji), swinging of hanging objects like the electric lamps, moving of liquids in vessels.
IV; Strong: Strong shaking of houses and buildings, overturning of unstable objects, spilling of liquids out of vessels.
V; Very Strong: Cracks in the walls, overturning of gravestones, stone lanterns etc., damaging of chimneys and mud-and-plaster warehouses.
VI; Disastrous: Demolition of houses less than 30%, landslips, fissures on the roads, the ground etc.
VII; Very Disastrous: Demolition of houses more than 30%, intense landslips, large fissures in ground, the faults.

THE MEANING OF SPECTRA OF EARTHQUAKE RECORDS OBTAINED IN OR NEAR STRUCTURES

Merit P. White*

The response spectrum of a disturbance, for example the acceleration recorded in a particular location at a given time, is defined as the relation between the maximum response of a simple oscillator to the disturbance and the period or frequency of that oscillator. The oscillator may be undamped or may have a particular kind and magnitude of friction or damping. Different damping relations give different response spectra, of course. The response may be in terms of maximum displacement, deflection, velocity, acceleration.

Numerous earthquake response spectra have been constructed from accelerograms recorded during strong or moderate earthquakes. On account of the complexity of earthquake motions the spectrum has appeared to offer the simplest means of comparing earthquakes with respect to their engineering effects and of drawing general conclusions as to these effects.

All these spectra, especially those with zero damping, have contained a number of peaks and valleys, implying that small changes of oscillator frequency result in large changes in oscillator response. In about 1938 it was suggested verbally by Prof. Hugo Benioff of the California Institute of Technology that these peaks, or some of them at least, might not be really characteristic of the earthquake itself but rather contributions from the structures that house the recorders to the motion felt by the recorders. Partly for convenience and partly in the hope of correlating ground motions with the response of large buildings, accelerometers have frequently been placed in the basements of large, important structures. It is the suggestion that the motion recorded in such a place would be the combination of the motion of the earthquake together with the motion of the foundation relative to the earth. Presumably, this latter component, although small, would be nearly periodic and contain a number of cycles of period corresponding to the natural periods of the housing structure. The contribution to the total spectrum, especially undamped, might be important. These suggestions were contained in a report by the author that was prepared at the California Institute of Technology for the Department of Building and Safety of Los Angeles County, California in 1939.

To get evidence on this question some forced vibration tests were made at that time under the direction of Dr. D. S. Carder of the U. S. Coast and Geodetic Survey to determine the vibration characteristics of

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two Los Angeles buildings that housed USCGS accelerometers, the Subway Terminal Building and the Hollywood Storage Building. The former is a complicated structure with a large number of low order modes and frequencies corresponding to oscillations of different parts. The Hollywood Storage Building is a rectangular, nearly symmetrical structure 51 ft x 217 ft in plan and 141 ft (14 stories) high, with reinforced concrete walls and floors. It is an attractive structure for analysis.

The buildings were caused to oscillate in various modes by means of a mechanical vibrator capable of applying a periodic horizontal force of adjustable frequency and magnitude at a given point. The displacements so produced were small compared to those of a strong earthquake, the maximum being about 0.075 mm. In the test of the Hollywood Storage Building, motions were recorded at various points throughout the structure, including the basement where an accelerometer was permanently located, and outside the structure at distances up to several hundred feet. In the basement, the induced motion was found to have approximately 4.5 percent of the amplitude of the roof motion. There was a lag of about 65 in phase. At a distance of 100 feet the amplitude ratio was about 0.9 percent with a phase lag of about 108 degrees and at 500 - 750 feet the amplitude ratio was about 0.6 percent.

From these results it appears probable that the records obtained by the basement accelerometer in the Hollywood Storage Building contained not only components furnished by the motion of the building itself, but also may have received contributions from other large buildings in the vicinity. This would then be true for records obtained in other large buildings also. The question is what is the importance of this effect?

As a matter of interest, it should be mentioned that O'Hara has recently suggested that while the spectrum peaks are associated with the actual modes and frequencies of a complete system that are (a) excited, and (b) impose motion on the support of the recording device, the valleys of such a spectrum tend to occur at frequencies that would be found if that part of the system below the recorder (in this case the earth) were frozen (1). In other words, the structure acts as a damper, suppressing or reflecting ground energy at those frequencies equal to its own frequencies if rigidly supported. This correspondence between spectrum minima and fixed-support frequencies is not likely to be perfect on account of the probability of coincidence between a real frequency and a fixed-support frequency of different order.

Consider now the numerical results of the Hollywood Storage Building shaker tests and apply them to the accelerometer records obtained in basement and penthouse during the October 2, 1933 earthquake. It must be mentioned that the ratio of earthquake amplitude to shaker motion at the roof was of order 100 (7.5 mm compared to 0.075 mm.) in the fundamental E-W mode. This ratio is so large that one must be cautious in applying the shaker results blindly to predictions of earthquake response; however, it is believed that one can draw intelligent conclusions as to orders of magnitudes of effects found at the larger amplitudes. For example, the shaker indicated a fundamental E-W mode period of 0.5 seconds, while the corresponding period during the earthquake was about 0.6 seconds.

Figure 1 shows a sketch of the roof level accelerations in the E-W direction recorded during the October 2, 1933 shock. Note that the fundamental mode is prominent and contains some 15 - 20 oscillations with a maximum amplitude of order 60 cm/sec^2, lasting for about 10 seconds. Assuming that the foundation participation factor that was found by the shaker - 0.045 - holds for these much larger amplitudes, we would expect that the foundation accelerometer record would contain about 15 cycles of a maximum amplitude of about 2.7 cm/sec^2. These are too small to be evident in the record (not shown here). However, the contribution of this motion to the base acceleration spectrum can be easily estimated. For an acceleration $j_o \sin \omega t$ that continues for n cycles the undamped acceleration spectrum A_s has at resonance an amplitude that is approximately

$$A_s = n \pi j_o$$

In the present case, let $n = 15$, $j_o = 2.7$ cm/sec^2, $\omega = 2\pi/0.6 = 10.5$. Then the acceleration spectrum amplitude due to this motion is 126 cm/sec^2 or 0.13 g.

An acceleration spectrum has been determined for the recorded E-W basement acceleration and is shown in Figure 2. It can be seen that there is a peak at the period of the fundamental building mode (0.6 seconds), with a value of about 0.2 g. In determining this peak the time of occurrence was found to be about 9 seconds, or 15 building periods after the motion began. The author believes that about half of this total value, or the amount by which the peak exceeds the spectrum for slightly shorter periods is due to coupling between building and foundation. It does not follow that all spectrum peaks have this explanation but the author recommends that caution be used in drawing conclusions from earthquake spectra until something more is known about this.

Bibliography

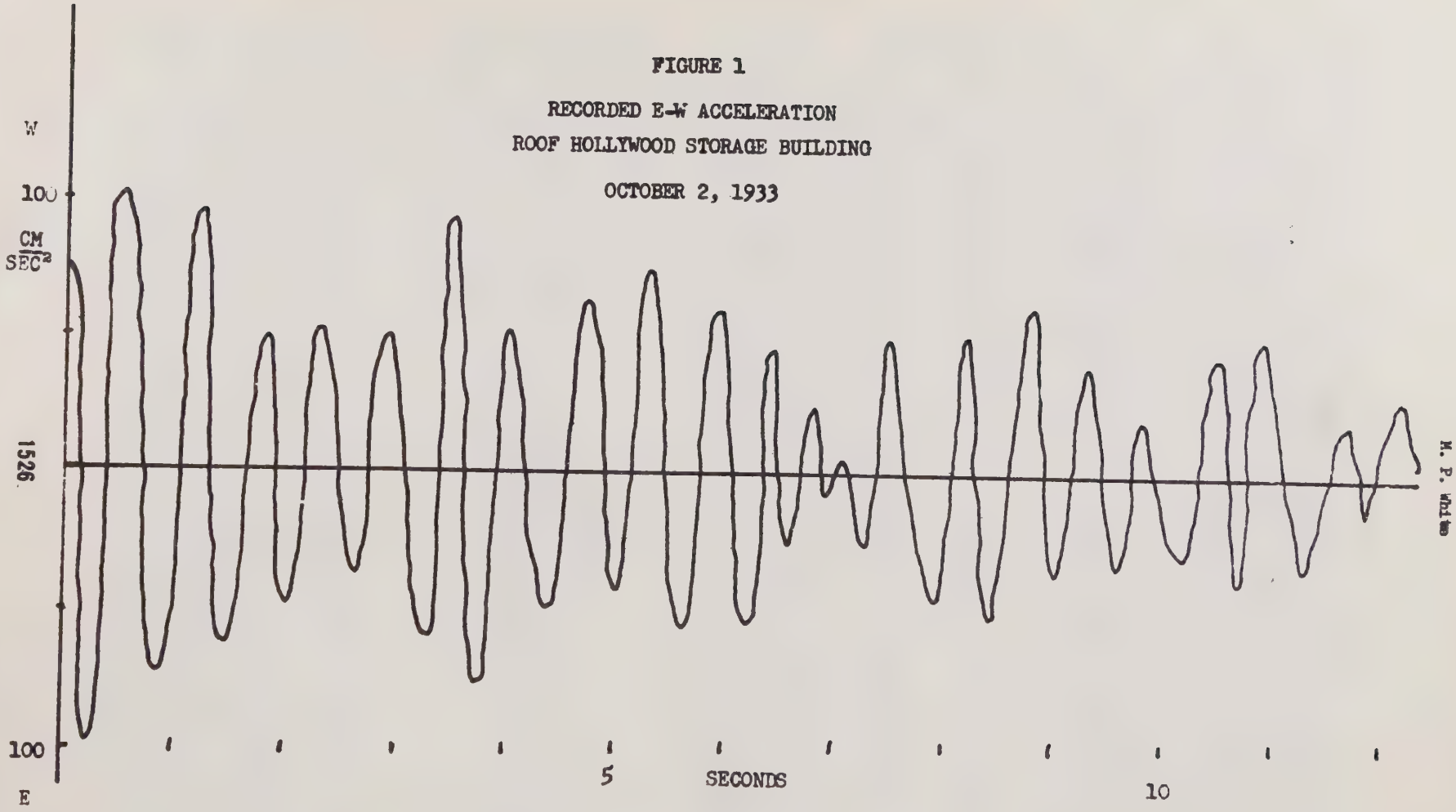
(1) "Effect upon Shock Spectra of the Dynamic Reaction of Structures," (NRL Report 5236) by George J. O'Hara, 16 Oct. 1958.

FIGURE 1

RECORDED E-W ACCELERATION

ROOF HOLLYWOOD STORAGE BUILDING

OCTOBER 2, 1933



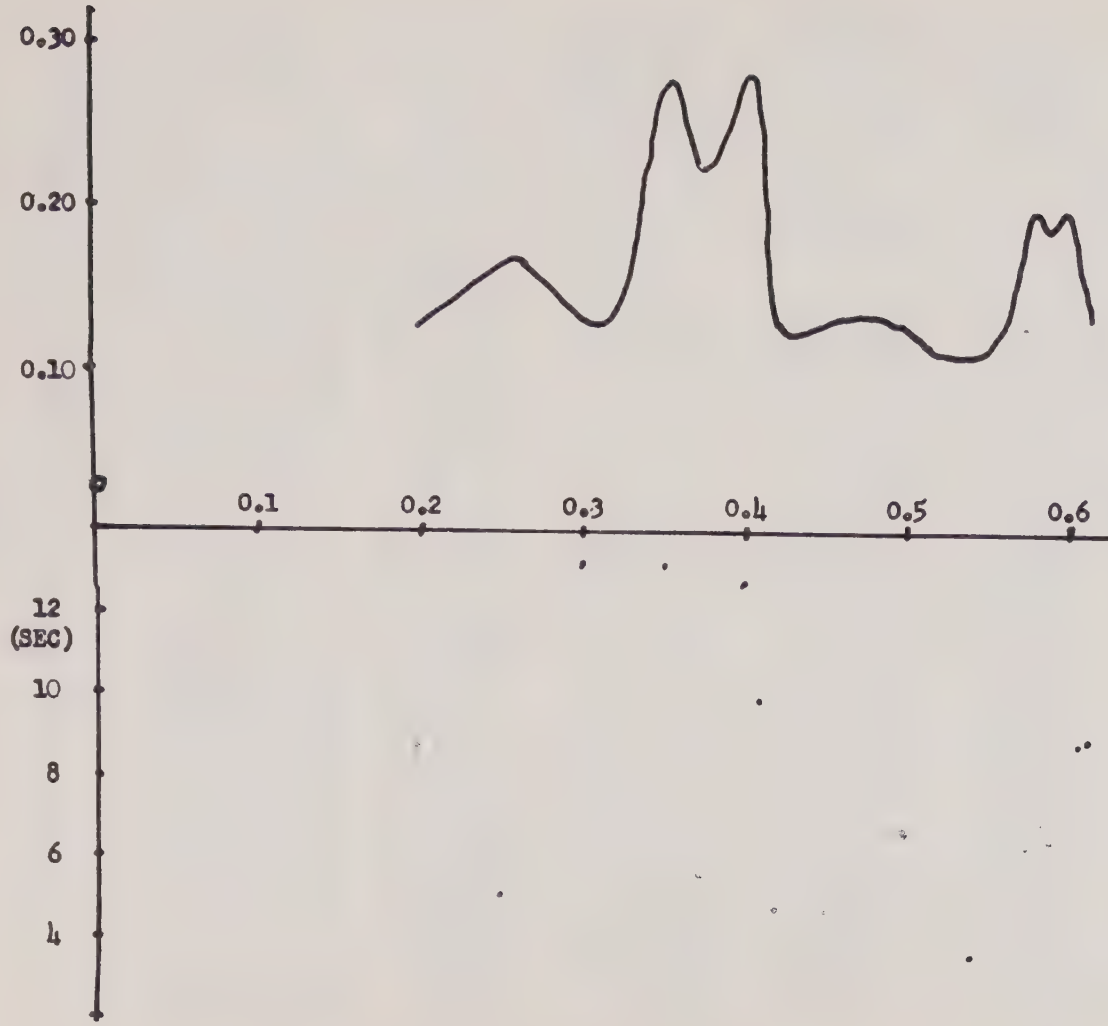


FIGURE 2

EQUIVALENT ACCELERATION IN g's

(ZERO DAMPING)

E-W HOLLYWOOD STORAGE BUILDING

OCTOBER 2, 1933

DISCUSSION

D. S. Carder, U.S. Coast & Geodetic Survey, U. S. A.:

Is the fundamental period from the earthquake (Oct. 1933) the same as that of normal vibrations set up by the wind?

M. P. White:

No, it is different. The fundamental period in the E.W. direction found in the October 1933 earthquake was 0.6^{s}, while the period observed in the shaker test was 0.5^{s}.

SEISMIC PROVINCES OF CHILE

by

E. Gajardo* and C. Lomnitz*

ABSTRACT

The seismic provinces of Chile are defined by using the statistical method of C. Tsuboi. Several thousand shocks collected during fifteen years were used in the computation. Tables and maps showing the relative seismicity of the regions are given.

INTRODUCTION

The earthquake distribution in Chile shows certain known geographic peculiarities [1]. The frequency of earthquakes is not evenly distributed but shows several peaks, the most prominent among which occur at Latitudes 28° and 32° South. It has also been established that the magnitude-frequency relation varies slightly for different regions of the country.

These preliminary results made it seem worthwhile to attempt a definition of seismic regions in Chile. Apart from its scientific value such a study was expected to provide information for the revision of the present earthquake provisions in the Building Code. These regulations calling for uniform design factors over the entire territory of Chile are being recognized as unrealistic and wasteful by many engineers.

From the seismic point of view Chile is a part of the Circum-Pacific earthquake belt. Its overall seismicity ranks between that of California and that of Japan. Chile, however, is unique in that it spans 38 degrees of latitude, or 2 1/2 times as much as the combined states of Washington, Oregon and California which form the West Coast of the United States. It constitutes the world's longest continuous earthquake belt under a single flag.

It is hardly surprising to find the existence of several seismic zones in a region of this length. Yet the actual zoning of the country is not easy to achieve. In the United States a zoning has been accepted on the basis of active, intermediate and inactive regions. In Chile the entire country, at least down to Parallel 40° South, is active.

C. Tsuboi [2] has proposed the concept of "Earthquake Province" which fits the Chilean situation very well. As in Japan we have here a continuous earthquake belt, but within this belt there are clusters of earthquake foci the activity of which can be correlated not only in space but in time as well. We hope to show that these earthquake provinces exist in Chile and that they are probably based upon separate tectonic mechanisms. To this end we have applied the statistical method used by Tsuboi in the above reference.

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SEISMIC CORRELATION STUDY

Tsuboi defines a correlation coefficient as follows:

$$r(l,m) = \frac{\sum_n \{N(l,t) - \bar{N}(l)\}\{N(m,t) - \bar{N}(m)\}}{\sqrt{\sum_n \{N(l,t) - \bar{N}(l)\}^2 \sum_n \{N(m,t) - \bar{N}(m)\}^2}}$$

in which N(l,t) is the number of earthquakes in the l-th compartment during the t-th year and $\bar{N}(l)$ means the annual average number of earthquakes in the l-th compartment. Then r(l,m) represents the correlation between compartments l and m as to yearly seismicity fluctuations.

In order to apply this criterion we have divided Chile into 37 compartments of one degree of latitude each (fig. 1). The earthquake statistics are given in Table I for the period 1942-1958 and partial periods 1942-49 and 1950-58. The compartments have been numbered according to latitude. As the information is based on the number of reported felt earthquakes a sketch of population density is given in fig. 1. The number of larger earthquakes (Mercalli intensities 6 and up) is also given in order to show that the data are not appreciably influenced by the density of observers. The general similarity between frequency curves indicates that the relative seismicity between regions is largely time-invariant.

The annual number of earthquakes in each region is given in Table II. This table may be used to find the correlation coefficients r(l,m) according to the formula above. The results are given in Table III and shown graphically in fig. 2. The value of r(l,m) is found at the intersection of the l-th column and the m-th row.

ANALYSIS AND CONCLUSIONS

In the case of a continuous seismic belt with an even or random distribution of seismic activity the correlation coefficients are expected to vary as a function of the distance between compartments only. Figure 2 shows, to the contrary, that the high values of r(l,m) tend to cluster about certain latitudes. These are the Earthquake Provinces defined by Tsuboi. The four most important ones are:

	Earthquake Province	Latitudes
A.	PAMPA DEL TAMARUGAL	19° to 22° South
B.	ATACAMA PROVINCE	26° to 29° South
C.	CENTRAL CHILE (Combarbalá to Curicó)	31° to 35° South
D.	CHILE SUR (Chillán to Osorno)	37° to 40° South

The region of Arica (18°- 19° South) is probably part of another earthquake province extending towards the North. There are some indications that Provinces B and C tend to grade into each other. The boundary seems to be in the latitude of La Serena (about 30° South).

The average value of correlation coefficients within the provinces defined above is + 0.37. Outside the provinces it is only about + 0.04. Another manner to define the earthquake provinces is to list the correlation coefficients between adjacent compartments (Table IV). One sees immediately that high positive correlation coefficients within the provinces are separated by negative coefficents. South of Puerto Montt (Parallel 42° South) the correlation coefficients are uniformly zero or negative.

Let us now lump all earthquakes in each province together and find the correlation coefficient between provinces (Table V). We see at once that the correlation between adjacent provinces A-B and B-C is rather high. In the case of the Japanese earthquake provinces the highest coefficient between adjacent provinces was + 0.16. This might mean that the regional mechanism called "earthquake field" by Tsuboi extends to a much greater distance in Chile than in Japan, where the mechanism is more of a local character. In particular it will be observed that the correlation coefficient between provinces A and B is + 0.40 while the coefficient between B and C is only + 0.26 in spite of the fact that these two provinces grade into each other geographically. We conclude that Northern Chile (North of La Serena, Parallel 30° South) forms a super-province or tectonic region in the sense of the "earthquake regions" defined by Gutenberg and Richter [3]. Such a new division appears well justified on geological and geographical grounds alone.

In conclusion it may be stated that the result of the present analysis confirms and clarifies the known information about earthquake distribution and tectonics in Chile. While it is recognized that the number of earthquake regions found by this procedure depends on the choice of size for the compartments, we believe that the four provinces defined herein do represent the fundamental zones of concomitant seismic activity in Chile.

BIBLIOGRAPHICAL FOOTNOTES

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2. C. Tsuboi (1958), "Earthquake Province -- Domain of Sympathetic Seismic Activities," Jour. Phys. of the Earth 6, p. 35.

3. B. Gutenberg and C. F. Richter, "Seismicity of the Earth" (Princeton University Press, 2nd Edition, 1954).

Instituto de Geofísica y

Sismología

Publication No. 11

SEISMIC ACTIVITY BY COMPARTMENTS

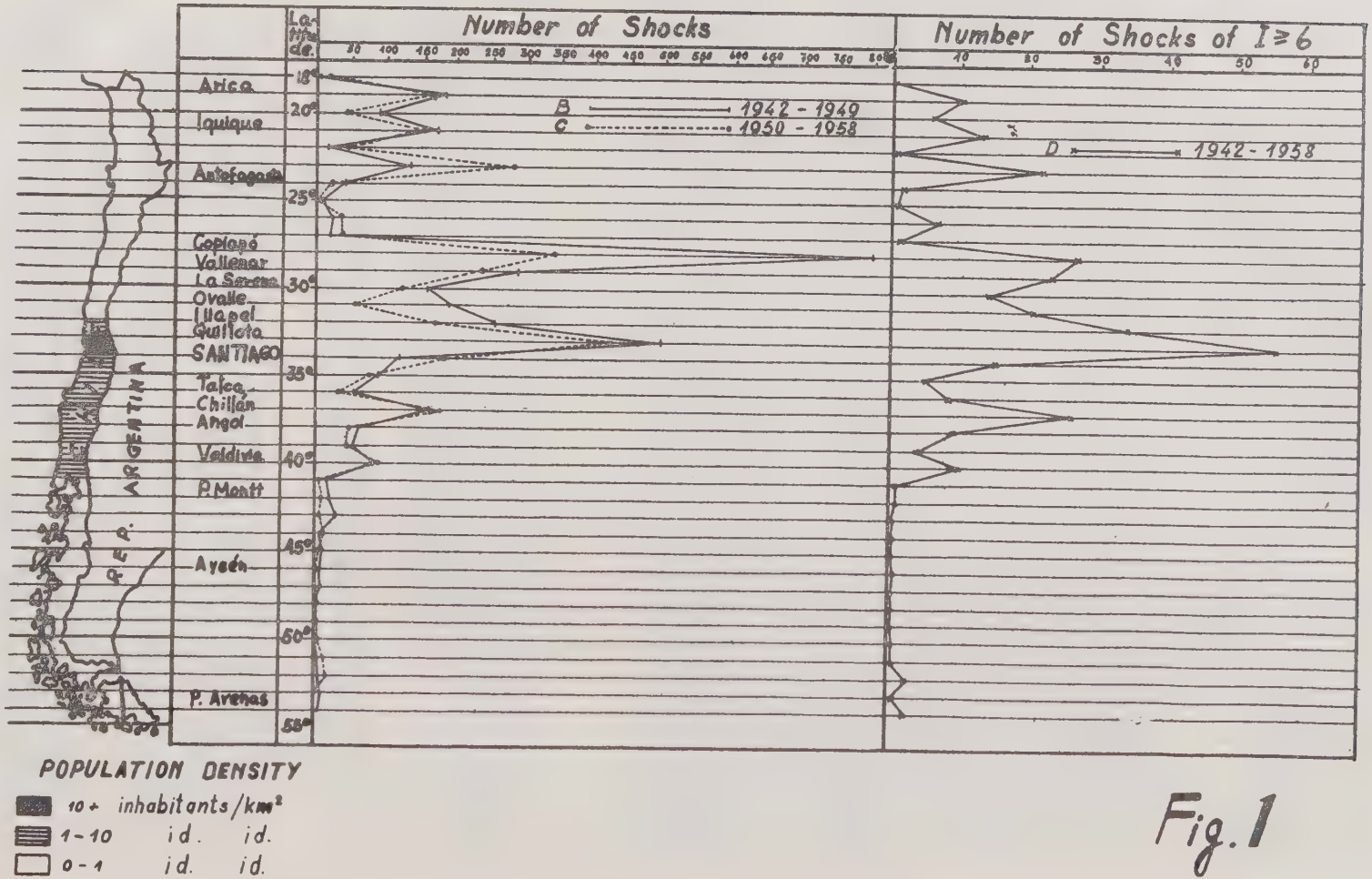

Fig. 1

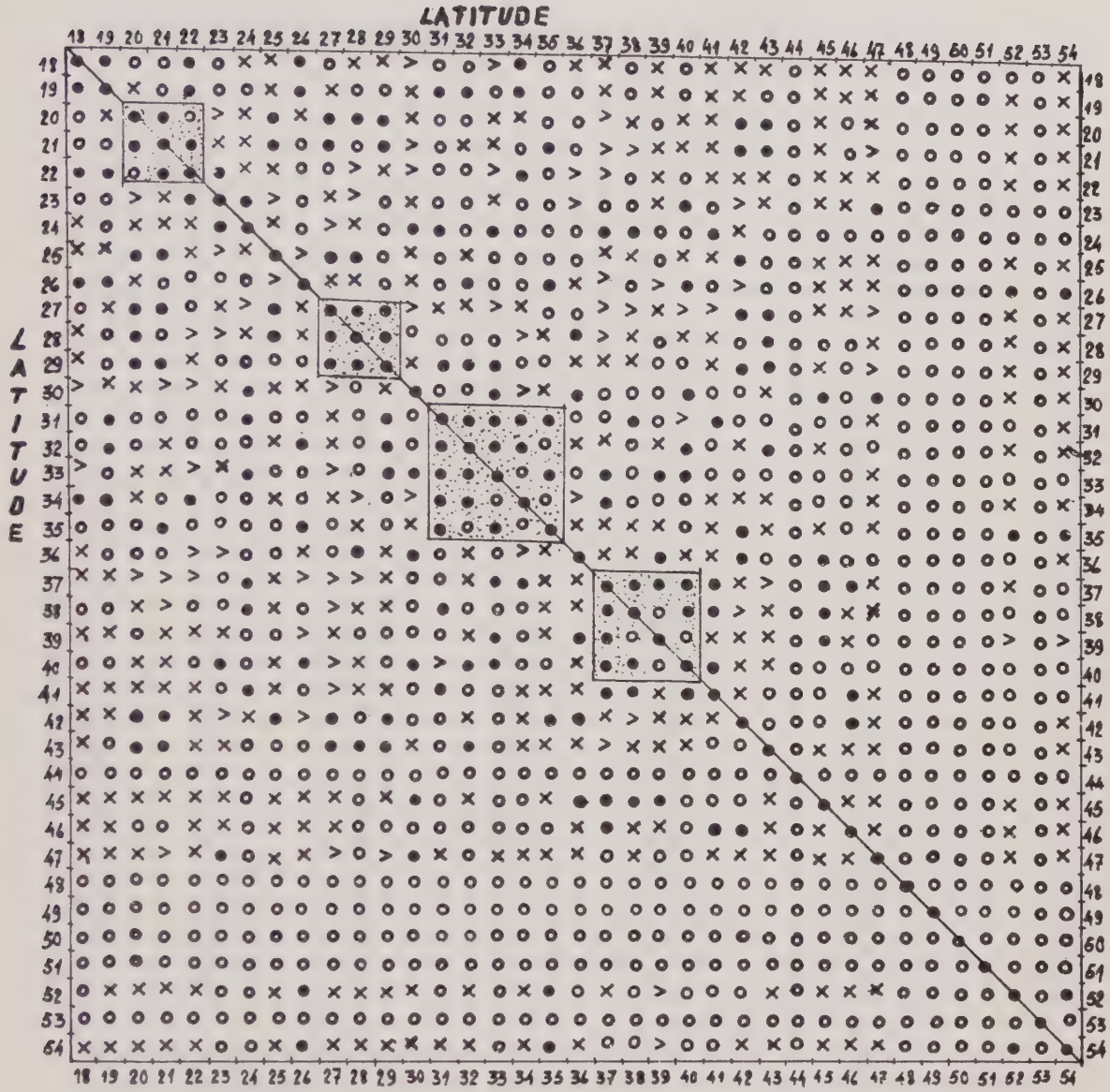
LATITUDE
LATITUDE
18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54
● ○ × >
≥0.3 <0.3 >-0.3 ≤-0.3

Fig. 2

TABLE I

Total number of earthquakes per Compartment

A 1942-1958
B 1942-1949
C 1950-1958
D 1942-1958
(shocks of Mercalli Intensity 6 and larger).

COMPARTMENT	A	B	C	D
18	1	0	1	0
19	345	167	178	10
20	113	83	30	6
21	337	172	165	13
22	39	2	37	0
23	415	135	280	22
24	62	39	23	2
25	12	12	0	2
26	53	25	28	6
27	52	25	27	2
28	1134	801	333	?27
29	528	289	239	23
30	278	153	125	13
31	249	190	59	18
32	431	258	173	34
33	950	522	428	55
34	288	121	167	15
35	196	90	106	5
36	93	60	33	ε
37	338	164	174	26
38	121	64	57	9
39	98	57	41	4
40	164	84	80	10
41	31	19	12	1
42	36	21	15	1
43	41	27	14	0
44	0	0	0	0
45	2	2	0	0
46	1	1	0	0
47	1	0	1	0
48	0	0	0	0
49	0	0	0	0
50	0	0	0	0
51	0	0	0	0
52	16	6	10	2
53	0	0	0	0
54	21	3	18	2

TABLE II

Annual Number of Earthquakes per Compartment

	1942	1943	1944	1945	1946	1947	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957	1958	I	II	III	IV	V	VI	
20°	18	52	16	18	16	10	34	19	18	15	15	11	14	17	6	21	1	1						1
	11			6	38	23	2	3			1					17	45	139	195	10	1			345
																	12	60	46	6	1			113
	28	10	2	29	43	25	20	15	17	22	7	5	6		16	59	33	179	145	13				337
	1				1							3	1	1	6	6	20	23	16	-				39
	22	34	10	21	5	5	12	26	28	7		30	45	45	44	47	34	191	200	22	1	1		415
25°	6	6	6	8		1	2	10	1	1		1	8	4	2	5	1	30	29	2	1			62
	-			1	10			1										1	8	2	1			12
	1	5	1	5		4	4	5	7	2	1	1	4	1	3	3	6	25	21	6	1			53
	1		3	1	8	4	6	2	3	3	1	3	2		3	8	4	18	31	2		1		52
	84	87	68	73	153	154	115	67	54	45	33	27	27	36	55	29	27	771	336	27				1134
30°	37	54	31	32	46	36	39	14	27	18	18	29	32	10	39	47	19	216	288	23	1			528
	23	25	28	21	13	15	14	14	10	13	24	1	18	29	18	8	4	124	135	13	6			278
	30	49	16	31	16	9	6	33	3	2	2	2	4	3	6	17	20	98	133	18				249
	27	95	14	15	16	48	23	20	18	8	9	19	15	29	20	26	29	211	184	34	1	1		431
	64	103	61	86	61	51	44	52	66	53	45	56	52	40	45	42	29	457	435	55	3			950
35°	20	28	17	17	9	9	9	12	12	21	14	28	13	11	12	24	32	163	108	15	1	1		288
	12	19	8	17	14	5	3	12	26	14	9	6	5	7	6	21	12	94	95	5	2			196
	6	3	11	9	8	6	12	5	7	3	6	2	2	6		4	3	41	44	8				93
	31	25	33	23	15	8	9	20	20	18	21	36	28	18	11	11	11	145	163	26	3	1		338
	5	12	12	6	5	5	3	16	4	2	6	8	10	7	6	6	8	62	48	9	1	1		121
40°	5	6	12	12	8	5	6	3	1	4	4	10	4	7	4	3	4	68	25	4	1			98
	13	16	12	14	6	6	5	12	9	4	3	7	12	13	12	9	11	87	67	10				164
	7	2	2	2		1	1	4	2		1	1	6	1			1	19	10	1	1			31
	4	2	3	2	6	2	1	1	3	3	2		1	1	1	3	1	12	23	1				36
	2	3	1	3	4	7	4	3	1	1		1	3	1	1	4	2	11	30					41
45°			2															1	1					-
																								2
	1													1				1						1
																			1					1
																								-
50°																								-
																								-
				1				5	9							1		2	12	2				-
								3	12			1	4		1			11	7	2	1			16
																								21
																								-
	459	636	369	453	510	439	374	377	358	259	222	288	316	288	317	421	369	3261	2836	316	27	6		6446

TABLE III

Correlation Coefficients by Compartments

	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
18	+1.00	+0.52	+0.13	+0.22	+0.92	+0.16	−0.21	−0.08	+0.37	+0.09	−0.25	−0.25	−0.40	+0.10	+0.05	−0.40	+0.53	+0.02	−0.19
19	+0.52	+1.00	−0.05	+0.11	+0.38	+0.14	+0.09	−0.10	+0.46	−0.04	+0.02	+0.23	−0.04	+0.58	+0.64	+0.25	+0.52	+0.28	+0.06
20	+0.13	−0.05	+1.00	+0.87	+0.16	−0.32	−0.27	+0.75	−0.20	+0.66	+0.63	+0.40	−0.26	+0.13	+0.04	−0.07	−0.15	+0.12	+0.17
21	+0.22	+0.11	+0.87	+1.00	+0.31	−0.05	−0.11	+0.40	+0.10	+0.72	+0.25	+0.43	−0.45	+0.22	−0.04	−0.11	+0.15	+0.46	+0.04
22	+0.92	+0.38	+0.16	+0.31	+1.00	+0.42	−0.23	−0.04	+0.26	+0.24	−0.37	−0.13	−0.43	+0.02	+0.01	−0.49	+0.57	+0.02	−0.39
23	+0.16	+0.14	−0.32	−0.05	+0.42	+1.00	+0.37	−0.32	+0.29	−0.14	−0.56	+0.06	−0.11	+0.09	+0.21	−0.10	+0.29	+0.13	−0.53
24	−0.21	+0.09	−0.25	−0.11	−0.23	+0.37	+1.00	−0.21	+0.22	−0.38	−0.11	+0.02	+0.38	+0.60	+0.13	+0.35	+0.02	+0.09	+0.04
25	−0.08	−0.10	+0.70	+0.40	−0.04	−0.32	−0.21	+1.00	−0.34	+0.50	+0.50	+0.28	−0.10	+0.09	−0.14	+0.11	−0.29	+0.12	+0.22
26	+0.37	+0.46	−0.20	+0.10	+0.26	+0.29	+0.22	−0.34	+1.00	−0.09	−0.07	+0.01	−0.28	+0.28	+0.30	+0.17	+0.11	+0.40	−0.08
27	+0.09	−0.04	+0.66	+0.72	+0.24	−0.14	−0.38	+0.50	−0.09	+1.00	+0.32	+0.41	−0.58	−0.23	−0.19	−0.37	−0.10	+0.07	+0.19
28	−0.25	+0.02	+0.63	+0.25	−0.37	−0.56	−0.11	+0.55	−0.07	+0.32	+1.00	+0.47	+0.12	+0.26	+0.29	+0.29	−0.48	−0.10	+0.49
29	−0.25	+0.23	+0.40	+0.43	−0.13	+0.06	+0.02	+0.28	+0.01	+0.41	+0.47	+1.00	−0.00	+0.38	+0.50	+0.48	+0.08	+0.19	−0.00
30	−0.40	−0.04	−0.26	−0.45	−0.43	−0.11	+0.38	−0.10	−0.28	−0.58	+0.12	−0.00	+1.00	+0.23	+0.19	+0.34	−0.34	−0.15	+0.31
31	+0.10	+0.58	+0.13	+0.22	+0.02	+0.09	+0.60	+0.09	+0.28	−0.23	+0.26	+0.38	+0.23	+1.00	+0.61	+0.64	+0.36	+0.40	+0.05
32	+0.05	+0.64	+0.04	−0.04	+0.01	+0.21	+0.13	−0.14	+0.30	−0.19	+0.29	+0.50	+0.19	+0.61	+1.00	+0.51	+0.32	+0.17	−0.19
33	−0.40	+0.25	−0.07	−0.11	−0.49	−0.10	+0.35	+0.11	+0.17	−0.37	+0.29	+0.48	+0.34	+0.64	+0.51	+1.00	+0.14	+0.55	+0.11
34	+0.53	+0.52	−0.15	+0.15	+0.57	+0.29	+0.02	−0.29	+0.11	−0.10	−0.48	+0.08	−0.34	+0.36	+0.32	+0.14	+1.00	+0.29	−0.48
35	+0.02	+0.28	+0.12	+0.46	+0.02	+0.13	+0.09	+0.12	+0.40	+0.07	−0.10	+0.19	−0.15	+0.40	+0.17	+0.55	+0.29	+1.00	−0.01
36	−0.19	+0.06	+0.17	+0.04	−0.39	−0.56	+0.04	+0.22	−0.08	+0.19	+0.49	−0.00	+0.31	+0.05	−0.19	+0.14	−0.43	−0.01	+1.00
37	−0.26	−0.13	−0.40	−0.52	−0.33	+0.02	+0.36	−0.14	−0.30	−0.51	−0.31	−0.07	+0.24	+0.17	−0.06	+0.46	+0.30	−0.01	−0.02
38	+0.07	+0.22	−0.24	−0.37	+0.02	+0.29	+0.68	−0.10	+0.16	−0.34	−0.15	−0.10	+0.20	+0.52	+0.28	+0.21	+0.18	−0.06	−0.15
39	−0.15	−0.10	+0.01	−0.22	−0.18	−0.21	+0.12	+0.19	−0.42	−0.09	+0.13	+0.11	+0.19	+0.12	−0.06	+0.34	+0.10	−0.20	+0.36
40	+0.09	+0.28	−0.10	−0.16	+0.12	+0.64	+0.21	−0.20	+0.30	−0.49	−0.16	+0.15	+0.40	−0.52	+0.42	+0.43	+0.23	+0.18	−0.18
41	−0.10	−0.01	−0.19	−0.23	−0.21	+0.16	+0.65	−0.20	+0.11	−0.46	−0.05	−0.09	+0.28	+0.33	+0.02	+0.23	−0.03	−0.06	−0.00
42	−0.20	−0.10	+0.64	+0.52	−0.23	−0.43	−0.15	+0.67	−0.33	+0.39	+0.15	+0.37	+0.43	+0.18	−0.08	+0.26	−0.16	+0.46	+0.43
43	−0.08	+0.10	+0.63	+0.49	−0.08	−0.12	+0.11	+0.25	+0.24	+0.45	+0.70	+0.47	−0.17	+0.28	+0.37	+0.08	−0.23	−0.06	+0.20
44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
45	−0.04	−0.09	−0.16	−0.29	−0.12	−0.23	+0.19	−0.07	−0.27	−0.00	+0.00	−0.02	+0.37	+0.03	−0.14	+0.07	+0.00	−0.14	+0.43
46	−0.07	−0.05	+0.10	+0.14	−0.06	−0.04	+0.19	−0.07	−0.25	−0.23	+0.10	+0.12	+0.21	+0.28	+0.02	+0.11	+0.10	+0.01	+0.04
47	−0.07	−0+07	−0.16	−0.32	−0.06	+0.33	+0.02	−0.07	−0.25	−0.33	−0.19	−0.44	+0.40	−0.21	+0.04	−0.23	−0.20	−0.18	+0.04
48	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
49	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
50	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
51	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
52	+0.01	−0.01	−0.14	−0.05	−0.09	+0.08	+0.08	−0.06	+0.61	−0.07	−0.06	−0.25	−0.24	+0.02	−0.12	+0.12	−0.19	+0.52	+0.09
53	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
54	−0.11	−0.14	−0.06	−0.03	−0.15	+0.08	+0.00	−0.10	+0.54	−0.07	−0.17	−0.15	−0.24	−0.23	−0.17	+0.10	+0.23	+0.45	−0.04

TABLE III (CONTINUATION)

	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54
18	-0.26	+0.07	-0.15	+0.09	-0.10	-0.20	-0.08	0.00	-0.04	-0.07	-0.07	--	--	--	--	+0.01	--	-0.11
19	-0.13	+0.22	-0.10	+0.28	-0.01	-0.10	+0.10	0000	-0.09	-0.05	-0.07	--	--	--	--	-0.01	--	-0.14
20	-0.40	-0.24	+0.01	-0.10	-0.19	+0.64	+0.63	0.00	-0.16	+0.10	-0.16	--	--	--	--	-0.14	--	-0.06
21	-0.52	-0.37	-0.22	-0.16	-0.23	+0.52	+0.49	0.00	-0.29	+0.14	-0.32	--	--	--	--	-0.05	--	-0.03
22	-0.33	+0.02	-0.18	+0.12	-0.21	-0.23	-0.08	0.00	-0.12	-0.06	-0.06	--	--	--	--	-0.09	--	-0.15
23	+0.02	+0.29	-0.21	+0.64	+0.16	-0.43	-0.12	0.00	-0.23	-0.04	+0.33	--	--	--	--	+0.08	--	+0.08
24	+0.36	+0.68	+0.12	+0.21	+0.65	-0.15	+0.11	0.00	+0.19	+0.19	+0.02	--	--	--	--	+0.08	--	+0.00
25	-0.14	-0.10	+0.19	-0.20	-0.20	+0.67	+0.25	0.00	-0.07	-0.07	-0.07	--	--	--	--	-0.06	--	-0.10
26	-0.30	+0.16	-0.42	+0.30	+0.11	-0.33	+0.24	0.00	-0.27	-0.25	-0.25	--	--	--	--	+0.61	--	+0.54
27	-0.51	-0.34	-0.09	-0.49	-0.46	+0.39	+0.45	0.00	-0.00	-0.23	-0.33	--	--	--	--	-0.07	--	-0.07
28	-0.31	-0.15	+0.13	-0.16	-0.55	+0.15	+0.70	0.00	+0.00	+0.10	-0.19	--	--	--	--	-0.06	--	-0.17
29	-0.07	-0.10	+0.11	+0.15	-0.09	+0.37	+0.47	0.00	-0.02	+0.12	-0.44	--	--	--	--	-0.25	--	-0.15
30	+0.24	+0.20	+0.19	+0.40	+0.28	+0.13	-0.17	0.00	+0.37	+0.21	+0.40	--	--	--	--	-0.24	--	-0.24
31	+0.17	+0.52	+0.12	-0.52	+0.33	+0.18	+0.28	0.00	+0.03	+0 28	-0.21	--	--	--	--	+0.02	--	-0.23
32	-0.06	+0.28	-0.06	+0.42	+0.02	-0.08	+0.37	0.00	-0.14	+0.02	+0.04	--	--	--	--	-0.12	--	-0.17
33	+0.46	+0.21	+0.34	+0.43	+0.23	+0.26	+0.08	0.00	+0.07	+0.11	-0.23	--	--	--	--	+0.12	--	+0.10
34	+0.30	+0.18	+0.10	+0.23	-0.03	-0.16	-0.23	0.00	+0.00	+0.10	-0.20	--	--	--	--	-0.19	--	-0.23
35	-0.01	-0.06	-0.20	+0.18	-0.06	+0.46	-0.06	0.00	-0.14	+0.01	-0.18	--	--	--	--	+0.52	--	+0.45
36	-0.02	-0.15	+0.36	-0.18	-0.00	+0.34	+0.20	0.00	+0.43	+0.04	+0.04	--	--	--	--	+0.09	--	-0.04
37	+1.00	+0.41	+0.50	+0.31	+0.55	-0.02	-0.46	0.00	+0.39	+0.33	-0.05	--	--	--	--	-0.01	--	+0.10
38	+0.41	+1.00	+0.14	+0.56	+0.37	-0.31	-0.04	0.00	+0.35	-0.14	-0.00	--	--	--	--	+0.11	--	+0.00
39	+0.50	+0.14	+1.00	+0.14	-0.10	-0.03	-0.05	0.00	+0.47	-0.07	+0.08	--	--	--	--	-0.43	--	-0.44
40	+0.31	+0.56	+0.14	+1.00	+0.45	-0.17	-0.07	0.00	+0.15	+0.21	+0.21	--	--	--	--	+0.07	--	+0.04
41	+0.55	+0.37	-0.10	+0.45	+1.00	-0.01	+0.00	0.00	+0.02	+0.65	-0.10	--	--	--	--	+0.14	--	+0.24
42	-0.02	-0.31	-0.03	-0.17	-0.01	+1.00	+0.15	0.00	+0.15	+0.34	-0.18	--	--	--	--	+0.02	--	-0.01
43	-0.46	-0.04	-0.05	-0.07	+0.00	+0.15	+1.00	0.00	-0.21	-0.07	-0.22	--	--	--	--	-0.14	--	-0.19
44	0.00	0.00	0.00	0.00	0.00	0.00	0.00	+1.00	0.00	0.00	0.00	--	--	--	--	0.00	--	0.00
45	+0.39	+0.35	+0.47	+0.15	+0.02	+0.15	-0.21	0.00	+1.00	-0.03	-0.03	--	--	--	--	-0.10	--	-0.10
46	+0.33	-0.14	-0.07	+0.21	+0.65	+0.34	-0.07	0.00	-0.03	+1.00	-0.07	--	--	--	--	-0.09	--	-0.11
47	-0.05	-0.00	+0.08	+0.21	-0.10	-0.18	-0.22	0.00	-0.03	-0.07	+1.00	--	--	--	--	-0.09	--	-0.11
48	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
49	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
50	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
51	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
52	-0.01	+0.11	-0.43	+0.07	+0.14	+0.02	-0.14	0.00	-0.10	-0.09	-0.09	--	--	--	--	+1.00	--	+0.89
53	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
54	+0.10	+0.00	-0.44	+0.04	+0.24	-0.01	-0.19	0.00	-0.10	-0.11	-0.11	--	--	--	--	+0.89	--	+1.00

TABLE IV

Correlation Coefficients r(m,m+1) between Adjacent Compartment

COMPARTMENT	r	TOTAL NUMBER OF EARTHQUAKES
18		1
19	+0,52	345
20	-0,05	113
21	+0,87	337
22	+0,31	39
23	+0,42	415
24	+0,37	62
25	-0,21	12
26	-0,34	53
27	-0,09	52
28	+0,32	1134
29	+0,47	528
30	0,00	278
31	+0,23	249
32	+0,61	431
33	+0,51	950
34	+0,14	288
35	+0,29	196
36	-0,01	93
37	-0,02	338
38	+0,41	121
39	+0,14	98
40	+0,14	164
41	+0,45	31
42	-0,01	36
43	+0,15	41
44	0,00	0
45	0,00	2
46	-0,03	1
47	-0,07	1
48	0,00	0
49	0,00	0
50	0,00	0
51	0,00	0
52	0,00	16
53	0,00	0
54	0,00	21
AVERAGE OF r	+0,17	

TABLE V

Correlation Coefficient between Seismic Provinces

	A	B	C	D
A	+1.00	+0.40	+0.02	-0.48
B	+0.40	+1.00	+0.26	-0.19
C	+0.02	+0.26	+1.00	+0.37
D	-0.48	-0.19	+0.37	+1.00

DISCUSSION

C. Tsuboi, University of Tokyo, Japan:

I have been much interested to know that the idea of "earthquake province" which I introduced in 1958 could be applied successfully to Chilean earthquakes also.

The difference which I have seen so far between Chilean and Japanese earthquake provinces is that the former is generally larger than the latter. I do not think this difference is caused by the difference in the compartment width used in these two studies: it is 110km for Chile and is 40km for Japan. I agree with Dr. Lomnitz's opinion that the difference in the average size of an earthquake provinces in Chile and in Japan represents something real.

S. Uyeda, University of Tokyo, Japan:

Your finding shows that the seismic regions in Chile seem definitely larger than those in Japan. Do you think this result is reality or would it be caused by your taking the size of each compartment larger than Prof. Tsuboi's in Japan?

C. Lomnitz:

In the case of Chile we took a compartment about 3 times as large as used by Prof. Tsuboi for Japan. If a tighter mesh size were used one would probably be able to determine several sub-regions within the four provinces defined herein. But the correlation coefficients within these provinces were rather high so I believe they do represent the basic four earthquake provinces in Chile.

D. S. Carder, U.S. Coast & Geodetic Survey, U. S. A.:

Is there any correlation between energy release and the number of shocks reported in the seismic zones that you showed on the first slide?

C. Lomnitz:

The correlation coefficient is a measure of time correlation of seismicity in a region. If the correlation coefficient is high, it means that the level of seismicity will be high or low at the same time within the region, but the actual value of the seismicity is not involved. Negative correlation coefficients indicate that if one zone becomes active the other is less active than normal.

AN EMPIRICAL FORMULA FOR THE SPECTRUM OF STRONG EARTHQUAKE MOTION

By KIYOSHI KANAI[1]

INTRODUCTION

The usual meaning of the term "earthquake motion" is the wave motion at ground surface transmitted from the earthquake origin through the earth's crust. If O(T), Q(T,q) and G(T,q) represent, respectively, the vibrational characteristics of the earthquake origin, of the earth's crust and of the ground at an observing station, the wave form of the earthquake motion at the station may be written as

$$u(T,q) = O(T) \cdot Q(T,q) \cdot G(T,q) \qquad (1)$$

in which T and q represent, respectively, wave period and wave attenuation. O(T) depends on both the earthquake magnitude and the mechanism of seismic wave generation, while Q(T,q) is determined by the reflections, refractions, scattering and absorption of waves in the earth's crust.

In this paper there is obtained first an empirical formula for the spectrum of earthquake motions at the ground surface, following which the formula is applied to certain cases for which strong motion earthquake observations are available.

THE SPECTRUM AT DEPTH

Earthquake motions at and near the surface are markedly influenced by the usually complex surface geologic conditions. Thus, in order to study analytically the characteristics of incident seismic waves, it is desirable to work with seismograms obtained at such depth that the surface geologic effects are negligible. The measurements obtained 300 meters underground at Hitachi Mine, 120 km. northeast of Tokyo, over a period of more than ten years, are well adapted to this purpose. All of the displacement-period curves obtained by spectral analysis of these seismograms exhibited a peak at some period.

Statistical analyses of earthquake records have shown that, as epicentral distance increases, the maximum displacement from the displacement-period curve decreases while the corresponding period changes only slightly[1)] Furthermore, the maximum displacement increases with the

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1) K. Kanai and S. Yoshizawa, "The Amplitude and the Period of Earthquake Motions. II", Bull. Earthq. Res. Inst., 36(1958), 275.
B. Gutenberg and C. F. Richter, "Earthquake Magnitude; Intensity, Energy, and Acceleration", Bull. Seism. Soc. Amer., 32(1942), 163.
A, E. Jones, "Empirical Studies of Some of the Seismic Phenomena of Hawaii", ditto, 28(1938), 313.

1 Earthquake Research Institute, University of Tokyo, Tokyo, Japan

corresponding period for a given epicentral distance.

The following empirical formula, which has been previously reported[2], describes the relation of maximum displacement to corresponding period of seismic waves at depth:

$$A_{ms} = 53\ T_m^{2.56} \tag{2}$$

in which A_{ms} in microns and T_m in seconds represent, respectively, the displacement and period coordinates of the peak of the displacement spectrum for a station at an epicentral distance of 100 km..

Except for periods much shorter or much longer than the above peak period, the spectrum of seismic waves at depth can be expressed as

$$\frac{\bar{u}}{T}\left(\equiv \frac{\text{velocity}}{2\pi}\right) = \text{constant} \tag{3}$$

in which $\bar{u}$ and T represent, respectively, the displacements and periods of individual points on the spectrum. This relation implies that seismic waves at depth are compatible over a considerable range of period with the concept of equipartition of energy among wave components. (Fig. 1 shows schematically the relations mentioned above.)

AMPLITUDE, PERIOD, MAGNITUDE AND EPICENTRAL DISTANCE

Many empirical formulae have been advocated to represent the relationship among magnitude, epicentral distance, and largest amplitude[3]. In the present case the following formula[4] is adopted:

$$\log A_m = M - 1.73 \log \Delta + 0.83 \tag{4}$$

where M is the Richter magnitude, Δ the epicentral distance in km., and A_m is the largest displacement amplitude in microns. In general, for long period waves such as are usually involved in (4), the largest amplitude at bed rock is approximately the same as that at the ground surface.

Considering a special case of (4) where $\Delta = 100$ km., and combining (4) with (2), there is obtained

$$\log T_m = 0.39\ M - 1.70 \tag{5}$$

From (4) and (5) we obtain the relation

$$\log \frac{A_m}{T_m} = 0.61\ M - 1.73 \log \Delta + 2.53 \tag{6}$$

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2) K. Kanai and S. Yoshizawa, loc. cit., 1).
3) B. Gutenberg and C. F. Richter, loc. cit., 1).
4) C. Tsuboi, "Determination of the Gutenberg-Richter Magnitude of Earthquakes occurring in and near Japan", Jour. Seism. Soc., Japan, (ii) 7, No. 3(1954), 185.

If the amplitude-period relation (3) is considered to be applicable to (6), we obtain

$$\bar{u} = T \times 10^{0.61\,M - 1.73 \log \Delta - 1.47} \tag{7}$$

which expresses the displacement amplitude $\bar{u}$ in cm. as a function of period of waves in seconds in bed rock.

It follows directly from (7) that

$$\bar{v} = 2\pi \times 10^{0.61\,M - 1.73 \log \Delta - 1.47} \tag{8}$$

$$\bar{a} = \frac{(2\pi)^2}{T} \times 10^{0.61\,M - 1.73 \log \Delta - 1.47} \tag{9}$$

where $\bar{v}$ is spectral velocity in cm./sec. and $\bar{a}$ is spectral acceleration in cm./sec.^{2}.

VIBRATIONAL CHARACTERISTICS OF GROUND

Understanding of the seismic characteristics of the ground surface has been becoming more clear as a result of observational investigations of earthquakes and microtremors, statistical analyses of earthquake damage, and theoretical studies of seismic waves. Based on earthquake and microtremor observations, and adding theoretical considerations, there has been obtained[5] a semi-empirical formula for the seismic characteristics of ground consisting of a layer overlying a semi-infinite medium:

$$G(T,q) = 1 + \left[\left[\frac{1+c}{1-c} \left\{ 1 - \left(\frac{T}{T_0} \right)^2 \right\} \right]^2 + \left\{ \frac{0.3}{\sqrt{T_0}} \left(\frac{T}{T_0} \right) \right\}^2 \right]^{-\frac{1}{2}} \tag{10}$$

in which T and T_0 represent, respectively, in seconds, the period of earthquake waves and the predominant period of the ground,

$$c = \sqrt{\frac{\rho_1 \mu_1}{\rho_2 \mu_2}},$$

and ρ_1, μ_1 and ρ_2, μ_2 are, respectively, the densities and elastic constants of the layer and the subjacent medium. (The expression(10) has been modified from that reported previously[5] in order to make it more closely analogous to the expression for motion of a pendulum relative to its support.)

The following special cases are presented as illustrations of (10).

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5) K. Kanai, "Semi-empirical Formula for the Seismic Characteristics of the Ground", Bull. Earthq. Res. Inst., 35(1957), 309.
K. Kanai, R. Takahasi and H. Kawasumi, "Seismic Characteristics of Ground", Proc. World Conf. Earthquake Engrg., Berkeley (1956), 31.

(i) The case of no surface layer and very long wave periods:

$T_o \longrightarrow 0$ $(T \longrightarrow \infty)$ and $G(T,q) \longrightarrow 1$.

(ii) The case of a thick surface layer and very short wave periods:

$T_o \longrightarrow \infty$ $(T \longrightarrow 0)$ and $G(T,q) \longrightarrow \frac{2}{1+c}$.

(iii) The case of resonance:

$T = T_o$, and $G(T,q) = 1 + \frac{\sqrt{T_o}}{0.3}$

In cases (i) and (ii) the ground magnifies the wave motion only very slightly. In case (iii) the values from (10) correspond very closely with those obtained from the previous version[6].

COMPARISONS OF COMPUTED AND OBSERVED SPECTRAL VALUES

Combining with equations (7), (8), (9) and (10) yields the following empirical formulae for surface ground motion spectra:

$$u = 0.034 \times T \times 10^{0.61\,M - 1.73 \log \Delta} \times \left[1 + \left\{\left[\frac{1+c}{1-c}\left\{1 - \left(\frac{T}{T_o}\right)^2\right\}\right]^2 + \left\{\frac{0.3}{\sqrt{T_o}}\left(\frac{T}{T_o}\right)\right\}^2\right\}^{-\frac{1}{2}}\right] \tag{11}$$

$$v = 0.213 \times 10^{0.61\,M - 1.73 \log \Delta} \times \left[1 + \left\{\left[\frac{1+c}{1-c}\left\{1 - \left(\frac{T}{T_o}\right)^2\right\}\right]^2 + \left\{\frac{0.3}{\sqrt{T_o}}\left(\frac{T}{T_o}\right)\right\}^2\right\}^{-\frac{1}{2}}\right] \tag{12}$$

$$a = \frac{1.34}{T} \times 10^{0.61\,M - 1.73 \log \Delta} \times \left[1 + \left\{\left[\frac{1+c}{1-c}\left\{1 - \left(\frac{T}{T_o}\right)^2\right\}\right]^2 + \left\{\frac{0.3}{\sqrt{T_o}}\left(\frac{T}{T_o}\right)\right\}^2\right\}^{-\frac{1}{2}}\right] \tag{13}$$

where u, v and a are respectively, the spectral displacement(cm.), velocity(cm./sec.) and acceleration(cm./sec.^{2}) at the ground surface.

The displacement, velocity and acceleration spectra, in the cases where M = 7 and 8 and Δ = 50 km., 100 km. and 150 km. besides the condition that c = 1/5 obtained by using equations (11)-(13) are shown in Fig. 3.

At Tokyo in the 1923 Kwanto earthquake there were obtained $T_o = 1.35$

6) K. Kanai, loc. cit., 5).

sec., T_o = 0.3 sec., M = 7.9, and Δ = 100 km.. The 1.35 sec. and 0.3 sec. periods were noted as corresponding to the predominant waves in the uptown area by Imamura[7] and Ishimoto[8], respectively. Applying (11), (12) and (13) the resonance values are as follows:

$$\left.\begin{aligned} T_o &= 1.35 \text{ sec.}; \; u = 5.1 \text{ cm.}, \; v = 24 \text{ cm./sec.}, \; a = 110 \text{ cm./sec.}^2 \\ T_o &= 0.3 \text{ sec.}; \; u = 0.66 \text{ cm.}, \; v = 14 \text{ cm./sec.}, \; a = 290 \text{ cm./sec.}^2 \end{aligned}\right\} \quad (14)$$

For comparison, the following values are estimated from the seismograms obtained by Imamura and Ishimoto:

$$\left.\begin{aligned} T_o &= 1.35 \text{ sec.}; \; u = 4.43 \text{ cm.}, \; a = 100 \text{ cm./sec.}^2 \\ T_o &= 0.3 \text{ sec.}; \; a > 250 \text{ cm./sec.}^2 \end{aligned}\right\} \quad (15)$$

The agreement between calculated and observed values is rather good.

The instrumental records and spectra[9] of eleven United States earthquakes recorded by the United States Coast and Geodetic Survey in the period 1933 to 1949 have been examined. The seismograph constants and earthquake data appear in Tables I and II. The resonance values of acceleration have been computed from (13), using as T_o the periods of the maximum trace accelerations from Table II. The calculated values are compared with values of maximum recorded accelerations in Table III and Fig. 2. (In Fig. 2 the two circles of different radii representing each observation station correspond with the two orientations of measurement of horizontal acceleration.) Again the agreement of calculated and observed values is good.

CONCLUSIONS

In determining earthquake ground motion characteristics for purposes of structural design, equations (11), (12) and (13) seem to be applicable. The magnitudes and epicentral distances of anticipated earthquakes may be estimated by the application of engineering judgment to statistical analyses of seismicity, and there can be thus obtained an approximate spectrum of seismic waves in bed rock. If this spectrum is combined with the vibration characteristics of the ground at the structure, the lateral force coefficient can be determined.

The author's many thanks are due to Professor C. M. Duke, University of California, Los Angeles, who has assisted him in preparing this paper.

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7) A. Imamura, "Report of the Investigation of the Kwanto Earthquake", Rep. Imper. Earthquake Inv. Com., No. 100A(1925), 27 (in Japanese).

8) M. Ishimoto, Kokin-Shoin, Tokyo (1925), 113 (in Japanese).

9) J. L. Alford, G. W. Housner and R. R. Martel, "Spectrum Analysis of Strong-Motion Earthquakes", 1st Tech. Rep. Office Nav. Res., Contr. No. 6onr-244, Task Order 25, Project Design, NR-081-091-(1951).

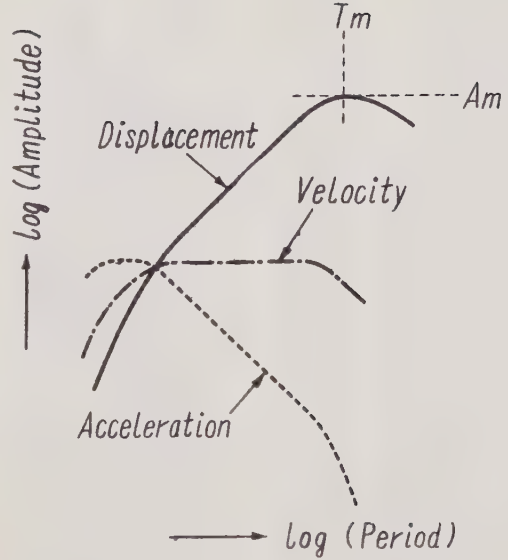

Fig. 1.
Schematic figures of the displacement, velocity and acceleration spectra of seismic waves at bed rock.

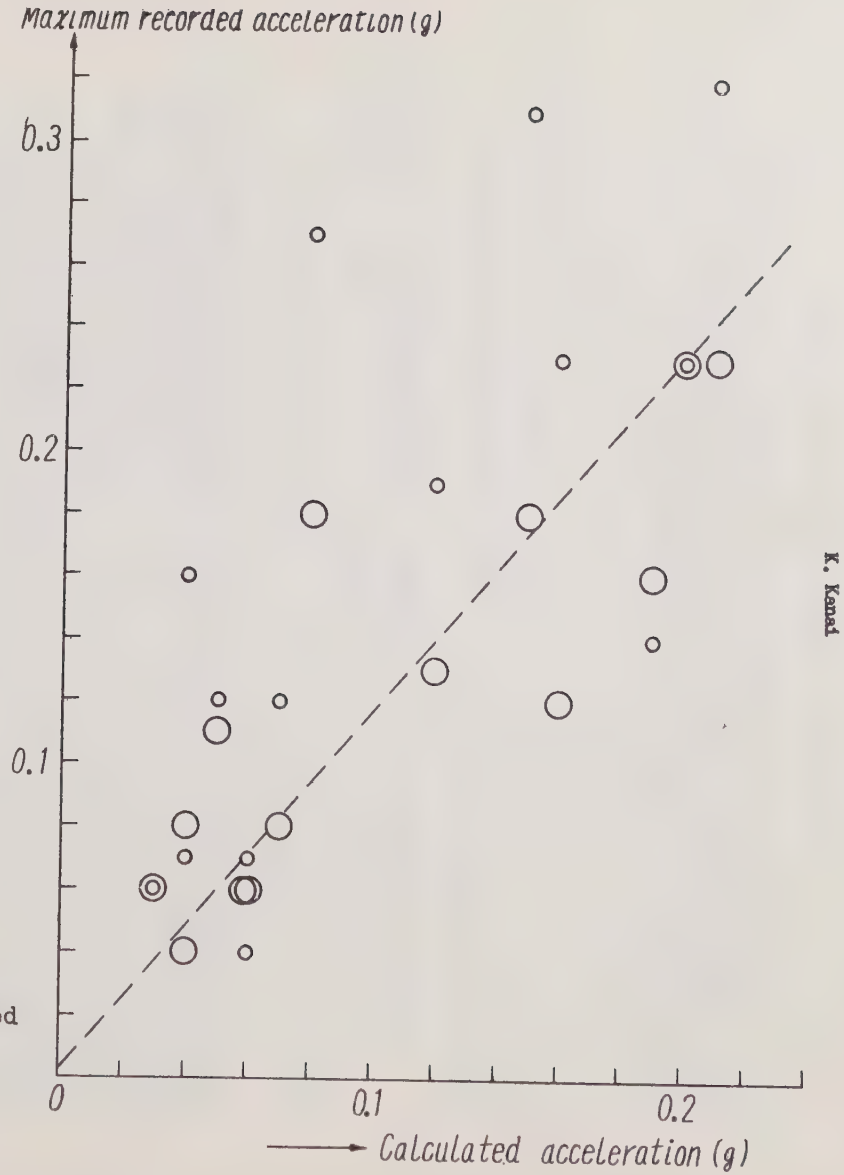

Fig. 2.
Relation between the maximum recorded accelerations and the values of calculated accelerations.

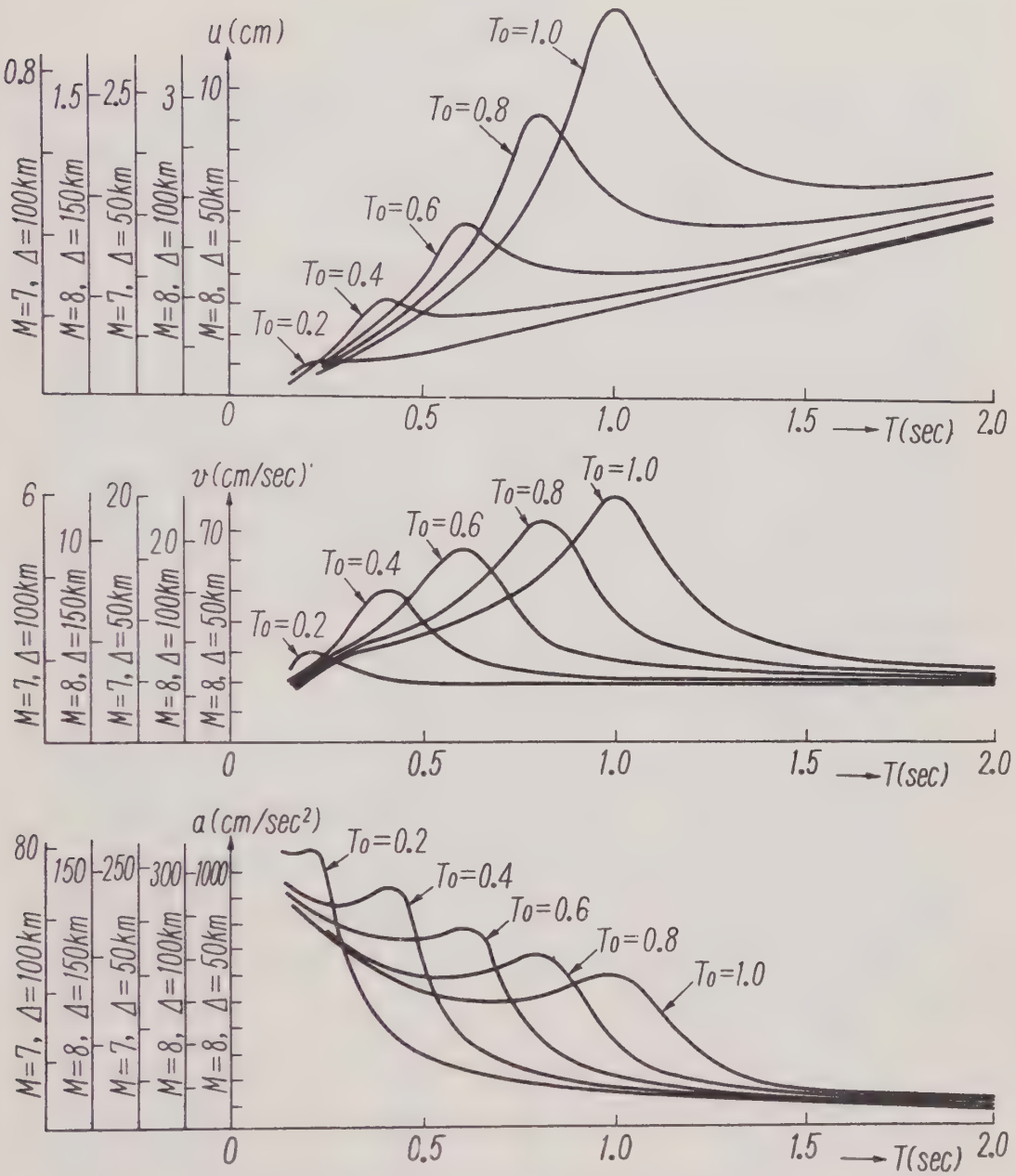

Fig. 3. The displacement(u), velocity(v) and acceleration(a) spectra obtained by using equations (11)-(13). T; period of waves, M; Richter magnitude, Δ; epicentral distance, T_0; predominant period of ground.

TABLE I

Instrumental Data

Station			Instrument		
No.	Location	No. of stories	Location	Orien-tation	Period (sec.)
I	Vernon, Calif. Centr. Manuf. Dist. Term.	6	Basement	S82E	0.065
				N08E	0.065
II	Los Angeles, Calif. Subway Terminal Bldg.	13	Subbasement	N51W	0.065
				N39E	0.065
III	El Centro, Calif. Irrig. Dist. Sub-station	1	Ground level, slab	NS	0.065
				EW	0.065
IV	Helena, Montana Carroll College	5	Basement	NS	0.080
				EW	0.080
V	Ferndale, Calif. City Hall	2	Slab	N45E	0.066
				S45E	0.065
VI	Santa Barbara, Calif. Court House	2-4	Basement	N42E	0.064
				S48E	0.065
VII	Hollister, Calif. City Library	1	Basement	S01W	0.066
				N89W	0.066
VIII	Olympia, Washington Hiway Test Lab.	1	Slab	S10E	0.081
				S80W	0.080
IX	Seattle, Washington Army Deport	1	Shed	N88W	0.085
				S02W	0.085

TABLE II

The Values of Periods

Earthquake			Station		Period (sec.)	
No.	Date	M	No.	△ (km.)	Max. accel. recorded	Max. accel. of spectrum
1	Mar. 10, '33	6.3	I	45	0.3	0.3
			II	53	0.65	0.65, 0.3
2	Oct. 2, '33	5.3	I	27	0.35	0.3
			II	35	0.65	0.65, 0.3
3	Dec. 30, '34	6.5	III	56	0.5	0.5, 0.2
4	Oct. 31, '35	6.0	IV	24	0.4	0.4, 0.25
5	Sept. 11, '38	5.5	V	56	0.2	0.2, 0.4
6	May 18, '40	7.0	III	48	0.5	0.5, 0.2
7	Feb. 9, '41	6.6	V	121	0.3	0.3, 0.6
8	June 30, '41	5.9	VI	24	0.3	0.3, 0.6
9	Oct. 3, '41	6.4	V	80	0.3	0.3, 0.6
10	Mar. 9, '49	5.3	VII	16	0.35	0.35
11	Apr. 13, '49	7.1	VIII	72	0.35	-
			IX	88	0.9	0.9, 0.3

TABLE III

The Values of Accelerations

Earthquake Number (Table II)	Station Number (Table I)	Orien-tation	Acceleration (g)	
			Observed	Calculated
1	I	S82E	0.19	0.12
		N08E	0.13	
	II	N51W	0.06	0.06
		N39E	0.04	
2	I	S82E	0.12	0.07
		N08E	0.08	
	II	N51W	0.06	0.03
		N39E	0.06	
3	III	NS	0.27	0.08
		EW	0.18	
4	IV	NS	0.14	0.19
		EW	0.16	
5	V	N45E	0.08	0.04
		S45E	0.16	
6	III	NS	0.32	0.21
		EW	0.23	
7	V	N45E	0.07	0.04
		S45E	0.04	
8	VI	N42E	0.23	0.20
		S48E	0.23	
9	V	N45E	0.11	0.05
		S45E	0.12	
10	VII	S01W	0.23	0.16
		N89W	0.12	
11	VIII	S10E	0.18	0.15
		S80W	0.31	
	IX	N88W	0.07	0.06
		S02W	0.06	

DISCUSSION

C. Tsuboi, University of Tokyo, Japan:

You have made use of my formula

$$\log A = M - 1.73 \log \Delta + 0.83$$

in your study. This formula was originally designed to give M of earthquakes in and near Japan from A and Δ data for these earthquakes observed in Japan. I wish to know if you have found that this formula works for earthquakes in America also. For instance, have you found agreements or disagreements between the Pasadena magnitudes of American earthquakes, and those determined by this formula?

K. Kanai:

From the result of the present investigation, it may be said that the agreements between the Pasadena magnitudes of American destructive earthquakes and those determined by your formula are rather good.

"SOME CONSIDERATION ABOUT THE EFFECT OF MEXICAN EARTHQUAKES"

By J. Figueroa A.

PERIODS

Abstract

Considering the seismograms of four stations (Mazatlan, Guadalajara-Tacubaya and Veracruz), each one located on terrains of different geological constitution (rock, conglomerate, shore-fillings), the period of the shear waves responsible for the maximum destructiveness has been measured.

Taking into account the epicentral distances from the origin to the limit of perceptibility in macroseisms of magnitude 7 and greater (Richter Scale), the predominant and most dangerous periods encountered were: 0.75-seconds, to 1.25 seconds, particulary those of 1 second.

The influence of focal depth is considered in order to confirm that the increase of the dimensions of the shaken block can cause destruction in soft grounds far from the epicenter.

The calculated magnitudes are compared with the grades of intensity felt in Mexico City.

We refer to the predominant periods of Normal Microseismic Agitation-related to meteorological phenomena and the periods of the Continuous Local Microseisms which vary with urban activity.

When a strong shock takes place it is assumed that the transverse --- waves, in most cases, lead along the maximum seismic energy, and these waves, more than others, produce the gratest damage.

Under that consideration we made an attempt in order to know the periods connected with such effects.

Four of our stations installed on different ground foundations were chosen because of the important role that the soil plays in the intensity of the earthquakes.

The stations and their instrumental constants are shown in table # I.

We did a careful selection of many seismograms obtained in these stations corresponding to the epicentral distances indicated in table # II.

We prefer strong shocks of the magnitude 7 and greater (Richter ---- Scale), but when, it wasn't possible due to the fact that with domestic --

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(1) Chief of the Seismological Service. Institute of Geophysics. National University of Mexico.

(nearby) shocks (100 Kms. or a little more epicentral distance), the re--- cording pens jumped up in many instances, making incomplete graphics, then we used seismograms of shocks of minor magnitude.

The obtained averages are shown in table II.

In this table we observe that until distances are smaller than 400 -- Kms. the period is proportional to the distance, that is to say, the lesser distance the lesser period.

Over 400 Kms. we find that the period diminishes as much as in epicen tral distances of 120 to 300 Kms.

We have found that prevailing periods in the greater part of seismo-- grams of macroseisms within the margin of distances encircled within the - limits of perceptibility are: 0.75 to 1.25 sec., particulary those of 1 -- sec., and for the finality of this work, we consider such periods as the - most dangerous.

Now then, in the course of the measurement it was confirmed that the- periods are shorter when the intensity of the earthquake diminishes.

On the other hand the periods are longer and the duration of the phe- nomenon lasts more, when the epicentral distance increases and the geolo-- gical constitution of the soil is poor (anelastic.)

In such type of soil, the natural decrement of the seismic energy by- the enlargement of the distance does not take place. On the contrary the-- values of the acceleration increase and it is obvious that if the duration increases, as we said, there will be more probabilities of resonances with the result of greater damages in badly designer or badly built construc--- tions, mainly when they are in soils with a high level of water.

In this kind of surface formations, of which we are talking about the seismic waves of shocks whose focal depth is not too shallow, have singu-- lar importance.

As a matter of fact any increment of depth means greater extension of the shaken block. This circunstance causes or is the cause that in poor -- grounds some times situated as far as 500 Kms. from the origin of an earth quake of magnitude 7.5 or greater, the intensity effects are compared only with those in the epicentral area. Heavy property damage and disasters --- have occurred.

We could mention many instances concerning Mexico City, but as a means of comparison two cases are enough:

The macroseisms June 7, 1911, destructive in Ciudad Guzman, Jal., had 100 Kms. of depth and an epicentral distance of 474 Kms. from Tacubaya (an environ of Mexico City). By this shock many persons were killed and inju-- red. Heavy property damage took place and some rails of street cars were - twisted, in the lower parts of the City of Mexico.

Now then, the source of the shallow strong shock of January 19, 1912- (magnitude 7.8, Tacubaya) was located near to Acambay and Tixmadeje. These two places in the State of Mexico only 100 Kms. far from Tacubaya were destroyed. However, in Mexico City the losses were negligible, in spite that the epicenter was closer in this earthquake than in the other.

The first earthquake was felt over an area of 500,000 Kms^2, while the second only reached 95,500 Kms^2.

After what has been said, it is useful to see in the next table III - how the Calculated Magnitudes of some earthquakes are compared with the -- grades of intensity felt in the Federal District.

That comparison shows us what ordinarly happens with shallow or deeper than normal shocks, and also confirm that the azimuth of the epicenter has not any influence in the grade of intensity.

Finally in a great number of seismograms of Tacubaya and Veracruz we- analize the inscriptions of predominant periods and amplitudes caused by- Normal Microseismic Agitation and Continuous Local Microseisms.

This agitation., as we know, are related: the former to the meteorological phenomena, the later to the urban activity.

The averages are shown in table IV.

From the former we realize that the local agitation in Tacubaya reaches periods a little more than 0.4 sec. while the amplitudes ascend at -- 10 h, 14 h, and 18 h, when there is a great deal of traffic.

Veracruz renders in general, greater periods, but smaller amplitudes, because its seismographs have lesser magnification; besides the traffic -- there, is not so intense as in Mexico City.

All what has been said, is a brief summary resulting from a great --- number of careful analysis and measurements made by three persons working- separately.

Conclusions: (according to expressed requirements.)

1 sec.	Predominant period in seismograms of stations located on hard ground.
1.22 sec.	Predominant period in seismograms of stations on less firm ground.
Longer periods	In stations on shore-fillings.
Amplitude	Greater in poor grounds.
Duration	" " " "

Acceleration Greater in poor grounds.

Tacubaya, D. F., February 29, 1960

J. Figueroa A.

TABLE # I

STATIONS AND SEISMOGRAPHS

NAME OF THE STATIONS	FOUNDATION	LATITUDE N	LONGITUDE W	HEIGHT (meters)	SEISMOGRAPHS
Tacubaya	conglomerate	19°24'18"	99°11'57"	2297	Wiechert horizontal:-- NS-EW, Mass 17000 Kg.- V=2000. To=1.5s. speed 60 mm m. NS-EW, Mass 1200 Kg. - V=250. To=6 s., speed- 30 mm m. Wiechert vertical Z, - Mass 1300 Kg. V=160.To =6 s., speed 30 mm m. Mass 80 Kg. V=80 To=8s speed 45 mm m. 2 Bosh-Omori horizon-- tal. NS-EW, Mass 10 Kg V=15. To=15 s., speed- 15 mm m.
Mazatlan	andesite	23°11'17"	106°24'22"	65	Wiechert horizontal,NS EW, Mass 200 Kg. V=80. To=6 s., speed 12 mm m Wiechert vertical Z, - Mass 80 Kg. V=80. To=4 s., speed 12 mm m.
Veracruz	shore-fillings	19°12'02"	96°08'16"	3	Wiechert horizontal, - NS-EW, Mass 200 Kg. V= 80. To=6 s. speed 15 - mm m. Wiechert vertical Z, - Mass 80 kg. V=80. To=4 s., speed 15 mm m.
Guadalajara (Tlaquepa-- que)	sedimentary rock	20°40'46"	103°19'27"	1567	Wiechert horizontal, - NS-EW, Mass 200 Kg. V= 80. To=6 s., speed 12- mm m. Wiechert vertical Z, - Mass 80 Kg. To=4 s. V= 80. speed 12 mm m.

N S: North-South component

E W: East-West component

Z: vertical component

V: magnification

To: period

mm m: millimetres per minute

TABLE # II

PREDOMINANT PERIODS

Epicentral - distance in- kilometres.	S T A T I O N S			
	MAZATLAN andesite rock	GUADALAJARA (Tlaquepaque) sedimentary rock	TACUBAYA conglomerate	VERACRUZ shore-fillings
	S E C O N D S			
15 Kms.		0.3 sec.	0.5 to 0.65 sec.	
50 - 70 Kms.	0.33 sec.	0.5 sec.	0.7 "	1.5 sec.
80-100-120Kms	0.66 "	0.7 "	0.76 "	1.7 "
120 - 300 "	0.75 "	0.75 "	1.0 "	2.0 "
300 - 400 "	0.75 "	1.25 "	1.22 "	2.16-2.31 "
400 - 500 "	Limit of perceptibility in shocks of --- magnitude 7.8		0.97 "	1.9

TABLE # III

CALCULATED MAGNITUDES COMPARED WITH THE GRADES OF INTENSITY FELT IN FEDERAL DISTRICT

PERIOD SEC.	DATE	DIST. Kms.	h Kms.	COORDINATES	MAG.	INT.	ACCELERATION mm/sec^2 TACUBAYA	ROSSI FOREL
0.97	1911 VI 7	474	100	19°40' N 103°39'W	8	VIII	250 - 500	500
0.95	1912 XI 19	95-119	Sup.	19°56'N 99°50' W	7.8	V	25 - 50	110
1.00	1920 I 3	216-235	Sup.	19°16'N 96°58' W	7.8	IV	10 - 25	80
0.97	1928 VI 17	408	50-80	16°13'N 97°11' W	7.5	VI	50 - 100	150
0.97	1932 VI 18	496	50-80	19°03'N103°54' W	7	VI	50 - 100	150
1.22	1937 XII 23	334	normal	16°24'N 98°39'W	7.2	VI	50 - 100	150
0.97	1941 IV 15	452	s.d.t.n.	18°07'N 103°19'W	7	VII	100 - 250	300
1.22	1957 VII 28	336	normal	16°21'N 99°13'W	7.5	VII	100 - 250	300

Dis.: distance

h: depth

mag.: magnitude (Richter Scale)

Int.: intensity (Mercalli Mod. 1931)

sec.: second

Kms.: kilometres

Sup.: superficial

s.d.t.n.: Slightly deeper than normal

TABLE # IV

NORMAL MICROSEISMIC AGITATION

RELATED TO THE METEOROLOGICAL PHENOMENA

STATION	MEAN AMPLITUDE	PERIOD
Tacubaya horizontal	0.38 millimetres	3.9 seconds
" vertical	0.32 "	3.7 "
Veracruz horizontal	0.31 "	4.9 "
" vertical	0.12 "	3.2 "

(The values of the vertical in Veracruz may be modified by the changes in temperature The seismographs operates in a room, and even under -- this influence, notwithstanding that it has a protection against such changes.

For this reason the recorder pen some times goes on with limited sensibility and the size of inscriptions are shorter.)

CONTINUOUS LOCAL AGITATION

WHICH VARY WITH URBAN ACTIVITY

	2h		6h		10h		14h		18h		22h	
	A	To	A	To	A	To	A	To	A	To	A	To
TACUBAYA:	.41	0.42	.74	0.42	.90	0.42	1.7	0.4	.84	0.42	.67	0.4
VERACRUZ:	.11	4.72	.26	4.9	.25	4.7	.26	4.9	.24	5.25	.20	4.65

h: hours

A: amplitude

To: period

DISCUSSION

C. F. Richter, Visiting Professor at the University of Tokyo, Japan:

This valuable paper shows that in explaining intensity distributions, we must consider the effect of path, as well as of ground at the point of observation.

Seismograms of the earthquake of 1912 at European Stations indicate focal depth of over 100km. Possibly this deep earthquake caused a shallow earthquake in the Acambay-Tixmadeje District. Seismograms at short distance may be due to this shallow shock.

J. Figueroa A.:

I am grateful to Dr. Richter's comment, and I want to record my complete agreement.

INVESTIGATION OF THE GROUND IN THE NORTHERN PART OF THE CITY OF YOKKAICHI IN THE NORTH ISE INDUSTRIAL DISTRICT

Nobuji NASU *

For the purpose of establishing a heavy industrial zone, the reclamation work is contemplated in a comparatively wide area along the northwestern coast of the Bay of Ise, that is, along the sea-shore extending over the city of Kuwana in the north, Village of Kawagoye in the middle and the City of Yokkaichi in the south. To furnish useful materials to this establishment plan, a comprehensive investigation was made in the nature of the ground and as well as on the sea-bottom. A Committee [1] was organized in August, 1958, by the specialists of seismology, geology, architecture and civil engineering in compliance with the request of the Yokkaichi City Office.

In October, 1957, the Resources Council of the Science and Technics Agency made a recommendation to the Government of Japan to put the plan of the ground investigations in practice in which a synthetic measurement of the ground nature should be made all over the main cities of Japan with the purpose of increasing the reliability in the designs of important facilities and sometime retrenching the expenditures, if possible. It was endeavoured that the present investigation made by the Yokkaichi Ground Investigation Committee meets well with this purpose.

The ground investigation may be divided into the following items.

a) Geological investigation.
b) Investigation of the tectonic changes.
c) Soil tests.
d) Observations of micro-tremors
e) Classification of the ground.
f) Plan of utilizing the ground.

In the following paragraphs, some summaries of the results obtained with respect to these items will be given.

(A) Geological investigation.

Generally speaking, the geologic formation in the Yokkaichi region is simple as compared with that in the Kuwana and Kawagoye regions [2]. In the low flat area, there can be thought to exist fault lines trending approximately in the N-S direction underneath the ground. To the east of this fault zone, the base ground was subsided and upon which alluvial deposits accumulated with a considerable thickness.

The formation and the stratification of the sand and soil layers were made clear [3] along the sea-shore from the results of the borings, of which 9 were made by the Yokkaichi Ground Investigation Committee and 21 by the Tokai Iron Manufacturing Co,Ltd., Fig.1. The pre-existing data for deep wells were also used. As most of the borings were stopped at a depth of about 40 m, the geologic formation deeper than this depth

* Nobuji NASU, Professor of the University of Tokyo, Member of the Earthquake Research Institute

was unable to be determined. In general, the stratification in this district is as below.

(1)	Upper sand and gravel layers	Alluvium
(2)	Upper clay layer	20 - 40 m thick
(3)	Lower sand and gravel layers	Alluvium
(4)	Lower clay layer	20 - 60 m thick
(5)	Tertiary	Upper Pliocene

(B) Tectonic changes [4]

Along the leveling-route running near Yokkaichi, the precise leveling surveys were made in the past. Fig. 2 shows the changes in the land-level along this route which occurred ; (I) in a period of 1895 - 1929, (II) in that of 1929 - 1931 and (III) in that of 1931 - 1947 (or 1948). In these changes, the height of the bench mark No.174.1 (at Atsuta) was assumed to be unchanged.

There can be seen no remarkable change in the land-level near Yokkaichi along the leveling route. Because, the route runs along the hill side which is comparatively apart from the centre of the City of Yokkaichi. Consequently, it is likely that the changes in this figure do not represent the actual changes which are prevailing with a considerable amount in the coastal region.

A fault may be deducible in the underground at a certain place between bench marks Nos.1461 and 1462, there discontinuous changes were observable,especially in period (III).

(C) Soil tests [5]

The first work of the soil investigation is to gather the data which are available for the nature and the distribution of the soils in this district. These data were very helpful for our investigations.

For the upper and lower sand layers, the standard penetration tests were made in the boring holes mentioned in (A). In this test,a tube(Raymond tube) is penetrated on the bottom of the hole by two feet by falling a weight of 140 lbs from a height of 30 inches to strike this tube at its upper end. The tube is 2 feet 8 inches long and 1 3/8 inches in diameter. During this penetration, the number of blow which is necessary to penetrate one feet, N , is recorded. This value of N is widely used as an index representing the relative density of the sand.

For the upper and lower clay layers,however, sixty samples were taken up by means of the thin-wall samplers during the boring work. For these samples, various tests were made in the laboratory such as, moisture-content, liquid-limit, plastic-limit, unit-weight, void-ratio, density of solid-particle, consolidation, mechanical analysis, compression test and shearing test. From the results of these tests, the characteristics of the soil could be made clear.

The upper sand layer is fairly compact and has a average strength. Fine sand is contained with very small percentages.

The upper clay layer is soft at shallow places, but it increases its strength with increasing depth. The clay has been proved to be normal and highly compressible.

The lower sand layer is generally well compacted, though there is local variation of the amount of content of the gravel and silt. As there are few data for the lower clay layer, its physical nature is not certain. But ,the layer is thought to be fairly consolidated and has a large shearing strength.

Among the physical constants which have been determined by our soil tests, the following constants will be useful for the engineering purpose.

1) Number of penetration

For upper sand layer : N = 15 (mean value),moderately compact.
For lower sand layer : N = 30 - 50, fairly compact.
N ≧ 100, when gravel is contained.

2) Angle of internal friction

For upper sand layer : 30°
For lower sand layer : 35°

3) Cohesion

For upper clay layer : C = 0.1 - 0.65 kg/ cm^2
For lower clay layer : C = 0.5 kg/ cm^2 or in some cases C = 1.0 kg/ cm^2.

4) Coefficient of compressibility

For upper and lower clay layers : $C_v = 20 - 60 \cdot 10^{-4}$ $cm^2/sec.$

5) Foundations of the pre-existing structures and their subsidence

As the nature of the soil has much influence upon the stability of the structure,an investigation was made upon the foundations of the pre-existing structures and their subsidence.

The buildings in this district are, in general, the reinforced concrete buildings shorter than the four-storied. Most of them are founded directly on the upper sand layer or supported by short piles driven in this layer. As these buildings are comparatively light in their weights, there is no damage which may be due to an unequal subsidence of the foundations.

However, in the manufactory buildings which bear heavy load extending over wide areas, there can be seen conspicuous subsidence which sometime impedes the function of the production equipments, so long as the buildings rest on the upper sand layer,even though they are built on the mat-foundations. The subsidence of such buildings is especially remarkable near the sea-shore.

On the other hand, in some of the buildings which have underground rooms and bear evenly distributed loads upon the foundations, only a slight subsidence can be seen.

The heavy equipments in the manufactories or the landing-stages are generally supported by the piles which reach the lower sand layer. Although they are of heavy weight, their subsidence and unequal settling are very slight.

Fortunately, a precise survey of subsidence could be carried out in the site of a manufactory where the buildings of various constructions were built one after another. Since the erection of these buildings, the subsidence was measured by the leveling survey referring to the points fixed on these buildings. The standard bench-mark was set on the ground fairly apart from these buildings. The survey was repeated in every three months. Fig. 3 shows the mean actual values of subsidence and the calculated values basing on the results of the soil tests.

In A and B are illustrated the subsidence of oil tanks. The foundations are constructed by replacing the superficial layer of a thickness of 4.5 m by sand so as to be supported directly by the upper sand layer. As the area of loading is comparatively wide, large stresses are produced in the upper clay layer and as a result, a remarkable subsidence is actually occurring.

C illustrates an example of the foundation supported by short piles driven in the upper sand layer. As the load on the foundation practically acts near the clay layer, a comparatively large amount of subsidence has been observed. In the calculation of the subsidence, it is assumed that a load of 6.9 kg/m^2 is distributed at a depth of the lower ends of the piles.

In D, is illustrated a foundation of the structure which has an underground room. In this case, the effective load which acts upon the clay layer is equal to the total weight minus the weight of the sand displaced. Therefore, the subsidence is very slight as compared with the case of the building which has no underground room. The subsidence-curve (D) is that of an underground water-tank supported by the piles driven in the clay layer. The load in this case is 12 ton/m^2. The bouyancy is estimated to be 14 ton/m^2. Therefore, no compressive force has been acting practically since the tank was constructed.

In E, a structure is shown which is supported by the piers constructed by the method of caisson. In this case, the subsidence is slight such that it measures 3 cm in the past four years and now practically stopped. This kind of piers proves to be safe against the unequal subsidence.

It is essentially important that the ground is not only safe against the load which acts on the foundation, but also against the subsidence which caused the damage upon the upper structure. Therefore, the design-load on the foundation, that is, the allowable bearing power must be chosen so as to satisfy these two conditions. If the various natures of the soil can be determined by the tests as in our present investigation, the amount of subsidence and as well as the allowable bearing power

may be calculated. The foundation may be determined by using the diagrams as shown in Fig.4. In these diagrams, the safety-factor for the shearing strength of the clay is taken as 3, which will be necessary for securing the foundation against the failure.

We have experienced, however, that the unequal subsidence of the building is remarkable when the subsidence of the building as a whole is considerably large. In the case of the pier foundation or the mat-foundation which is effective to distribute the load widely in the soil layer, the amount of unequal settling is in a small proportion to the total subsidence when it is compared with that in the cases of the isolated and belt foundations. Actually, it has been found that the limit of the settling of the building as a whole which causes no damage due to unequal subsidence is 10 cm in the isolated foundation and 20 - 30 cm in the case of the mat and the pile foundations.

(D) Observation of micro-tremors [6]

1) Micro-tremors on the land

The observation of the micro-tremors was conducted for three days from November 20 to 22, 1958. The points of observation were selected at 13 places in the City of Kuwana, 13 places in the Village of Kawagoye and 76 places in the City of Yokkaichi, Fig. 5 .

The instruments which were used in this observation were the electro-magnetic seismographs registering one horizontal component motion on the smoked papers. This type of seismograph was consisted of the transducer, low-frequency amplifier and pen-writing oscillograph. In ordinary cases, the magnification was adjusted to be about 8,000 for a period of 0.4 second of the ground motion. The value is the peak value of the magnification in the frequency chracteristic curve of this seismograph.

The seismographs were set in the observation-car. At the point of observation, only the transducer was installed on the ground and the record of tremors is made in the car. The electric power was supplied by the secondary batteries by means of the invertor. Thus, we could make observation at any place, even there was no commercial electric supply. The record of tremors at one place was made for three minutes.

The method of constructing the frequency curve of the micro-tremors is as follows :

The record of tremors lasting two minutes is selected out of the record of three minutes. For all the waves in this two-minute-record, times are determined between two consecutive points at which the waves pass across the zero-line. The period of the wave is defined as the time which is twice as large as the time thus determined. The accuracy of determining the period is 0.02 second.

Let T_1 be a certain period of the tremor. The periods which differ from this value by a small amount, ΔT_1 , are represented by T_1 . Then, the number of waves lying in an interval of $T_1 + \Delta T_1$ is taken as the frequency of occurrence of the waves of which periods are T_1. In this case, the value of ΔT_1 is adjusted in proportion to T_1. The specimen frequency-curves which were thus constructed are shown in Fig. 6 .

The amplitude of wave is determined by taking the mean value of the amplitudes of a certain number of successive waves which are recorded with comparatively large amplitudes.

2) Micro-tremors on the sea-bottom

The micro-tremors on the sea-bottom were observed at 16 points by means of the seismograph specially designed for the sea- observation. The transducer of this seismograph is contained in a brass case which has an electric heater of 80 watt/hour at its lower end, Fig. 7 . The pendulum of the seismograph is an inverted pendulum having a natural period of vibration of 1.0 second. The brass case is hung by brass rings in a water-proof container. The electric heater is made so as to be in the solid paraffin in this container. After this container is set down on the sea-bottom, the electric current is supplied to the heater to melt the paraffin enabling the pendulum to rest in a vertical position automatically. The exact position of the pendulum is detected by means of the electric contact between the pendulum and the brass case. If the pendulum is found to be in a good position, the electric current is cut off to cool the paraffin which fixes the brass case to the container.

The instruments were set on a small boat. At a point of observation, the boat is anchored to set the seismograph on the sea-bottom. At this time, wires and cables are necessary to be made loose as possible.

From the results of the sea-observations, the classification of the ground was made similarly to the case of the observation on the land. The points of observation on the sea-bottom are distinguished by dashes attached to their serial numbers, Fig. 5 .

It has been found that the characteristics of the tremors on the sea-bottom are very similar to those of the tremors which were observed on the land.

(E) Classification of the ground

1) First proposal of the classification

It has been known that the predominant period, the mean period and the largest period do not change their values whenever they are measured. Here, we must remember that the largest period is the one which is largest among those of the waves registered at a certain point by means of our seismograph which has a natural period of oscillation of 1 second. The physical meaning of this largest period is not yet made clear, but it is convenient to use this period in the classification of the ground practically.

In the case of the frequency-period curve in which the predominant period can be seen clearly, it is known that the value of the mean period agrees with that of the predominant period. Therefore, we may use the mean period instead of the predominant period.

In the classification of the ground, diagrams as shown in Fig. 8

are proposed to be used. As can be seen in these figures, the values of the largest periods are plotted against the mean periods which were determined for various points of observation. Border-lines are drawn in parallel to the ordinate and the abscissa of this figure to classify the points into four kinds, I , II , III and IV. The geological classification of the ground as is used, for example, in the seismic zoning map of Tokyo, is as follows :

Kind I, tertiary or other very hard ground (not commonly found near the ground-surface in Tokyo).

Kind II, consists mainly of diluvium or of the ground with gravel strata 5 meters or more in thickness, forming the plateau of the uptown of Tokyo. The river basins located in the plateau which have the alluvial deposits are designated as Kind III, whereas the basin which has thin deposit is designated as Special Kind II.

Kind III, consists of alluvium 5 meters or more in thickness, which can be distinguished from the ground of Kind II by bluff formation.

Kind IV, consists of alluvium 30 meters or more in thickness or of reclaimed land. An exception is made for such thick alluvium where a firm gravel stratum lies near the ground surface, the ground is classified as Kind III.

In designating the boundary of each classified area. administrative considerations have been taken into account.

As point No.61,for example, lies in a space corresponding to Kind II in the foregoing figure, the ground represented by this point belongs to Kind II. Point No.42 lies in a space which is intermediate between III and IV, the ground near this point may be classified as Kind III - IV.

2) Second proposal of the classification

We have experienced that in a region where the ground is so soft that it may be classified geologically as Kind IV (or Kind III,sometime), nevertheless there appear short periods of tremors as the predominant period as in the case of a hard ground. On the other hands, it is recognized that the amplitudes of the tremors are usually larger on the soft ground than on the hard ground. Thereupon, it is convenient to plot the largest amplitude of the tremor aginst the predominant period. Thus, the classification of the ground can be easily made sometimes.

In Fig. 9, the largest amplitudes are plotted against the predominant periods,both in logarithmic scales.

From the above two proposals, the classification of the ground could be made at 76 points as shown in Fig. 5 . Among these points, there are 3 points at which the disturbances due to machineries of nearby factories were so large that the records of tremors were complicated and the classification became uncertain. These points are represented by circles in this figure.

(F) Plan of utilizing the ground

From the results of the investigation so far made in the northwestern part of the Ise Bay, the following utilization of the ground is recommendable.

In the northeastern plain extending from Kawagoye to Kuwana, the underground formation is likely uniform, that is, there lie the upper sand layer, upper clay layer, lower sand layer and lower clay layer.

The uppermost sand layer which is 2 - 10 m thick has a moderate strength as the sand layer and is capable of supporting the short concrete buildings.

The upper clay layer is fairly thick, about 35 m near the mouth of River Nagara, but it becomes thin and about 1 m near the River Kaizo. Although the strength of this layer increases with increasing depth, the layer is generally so soft that it is not suitable for the foundation of any structure. The subsidence of the foundation will be liable to occur, if this layer is laid under a continuous loading due to structures built on it.

In general, the lower sand layer has a thickness of about 30 m, but a thickness of only 1 m near the River Kaizo. The geological age of this layer is uncertain whether it is the earlier alluvium or the later diluvium. The layer is well compacted and favourable to the foundation of the structure.

As the lower clay layer is of an older age than the former layer, its hardness is sufficiently large as we can believe that there will be no objection occurring in the structure, if the foundation is laid on this layer.

As mentioned above, the uppermost sand layer may be used for the foundation of the short building, but the region where this layer is very thin or lacking and the upper clay layer appears directly on the ground surface is not suitable for any structure even if such a region is reclaimed. There the usage of the ground is strongly limited to the specially light facility.

In the erection of the heavy structure or of the manufacturing facility which needs special precision, the foundation must be reached the lower sand layer. In the region lying to the southwest of the River Machiya, this layer lies at comparatively shallow places. Therefore, this region may be used for the heavy industries.

As the lower clay layer has a possibility of consolidation in a long time owing to the gradual loss of contained water, whether the layer is loaded or not, there is a fear of sinking of the ground at the places where this layer is thickly accumulated, for examples, along the coast near Kuwana and Kawagoye. In the design of structure, the extent of the subsidence should be taken into consideration.

As the present investigation was limited to find out the suitable soil layers for the foundation of the heavy industrial facility, the

borings specially made by our Committee stopped at comparatively shallow places (mostly at depth of 40 m), it is sufficient to determine the geological condition down to the tertiary layers. We are deeply impressed that the investigation which was made this time in the Yokkaichi district is very useful as the practical method of determining the nature of the ground, especially in the future ground investigations which will be made for the city-planning such as recommended by the Resources Council of the Science and Technics mentioned ahead of this paper.

Bibliography

1) Working-groups were organized by the members of the Earthquake Research Institute (N.Nasu, R.Takahasi, F.Kishinouye, H.Kawasumi and K.Kanai) , the Building Research Institute (K.Takeyama and Y.Koizumi), Geological Survey Institute (N.Miyabe and M.Shiki), the University of Nagoya (I.Matsuzawa) and the Kambe High School (H.Akamine). Report of the Committee (in Japanese with summary in English) was published in November, 1958.

2) Isao Matsuzawa : General description of the geology in the northern coastal region of the Bay of Ise. (Report of the Committee cited in 1).)

3) Hideo Akamine : Geology in the northern area of Yokkaichi. (ditto.)

4) Naomi Miyabe : Tectonic changes in the northern Ise district. (ditto.)

5) Yasunori Koizumi : Report of the soil tests. (Report cited in 1).)
   (a) Nature of soils.
   (b) Foundations of pre-existing buildings and their subsidence.
   (c) Allowable bearing power on the foundation and consideration of the design of foundation.
   (d) Lists showing the results of soil tests.

6) Kiyoshi Kanai and Nobuji Nasu : Observations of micro-tremors.(ditto.)
   (a) Forewords.
   (b) Observation of micro-tremors on the land.
   (c) Observation of micro-tremors on the sea-bottom.
   (d) Consideration of the results.
   (e) General description of the micro-tremors.

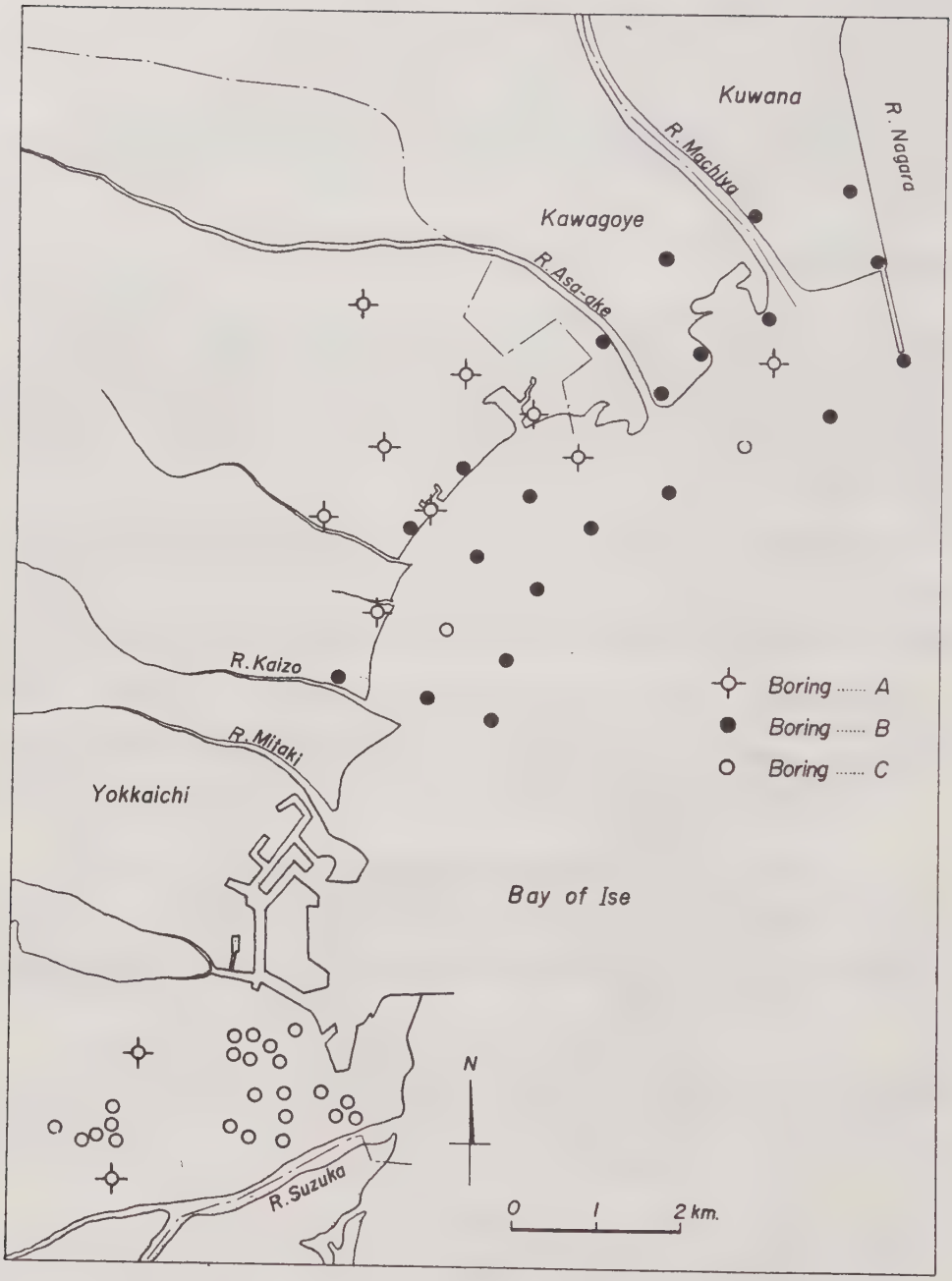

Fig. 1. Positions of borings.

Boring A....Sampling and standard penetration test were made.
Boring B....Penetration test was made only.
Boring C....Other borings of which data were used as reference.

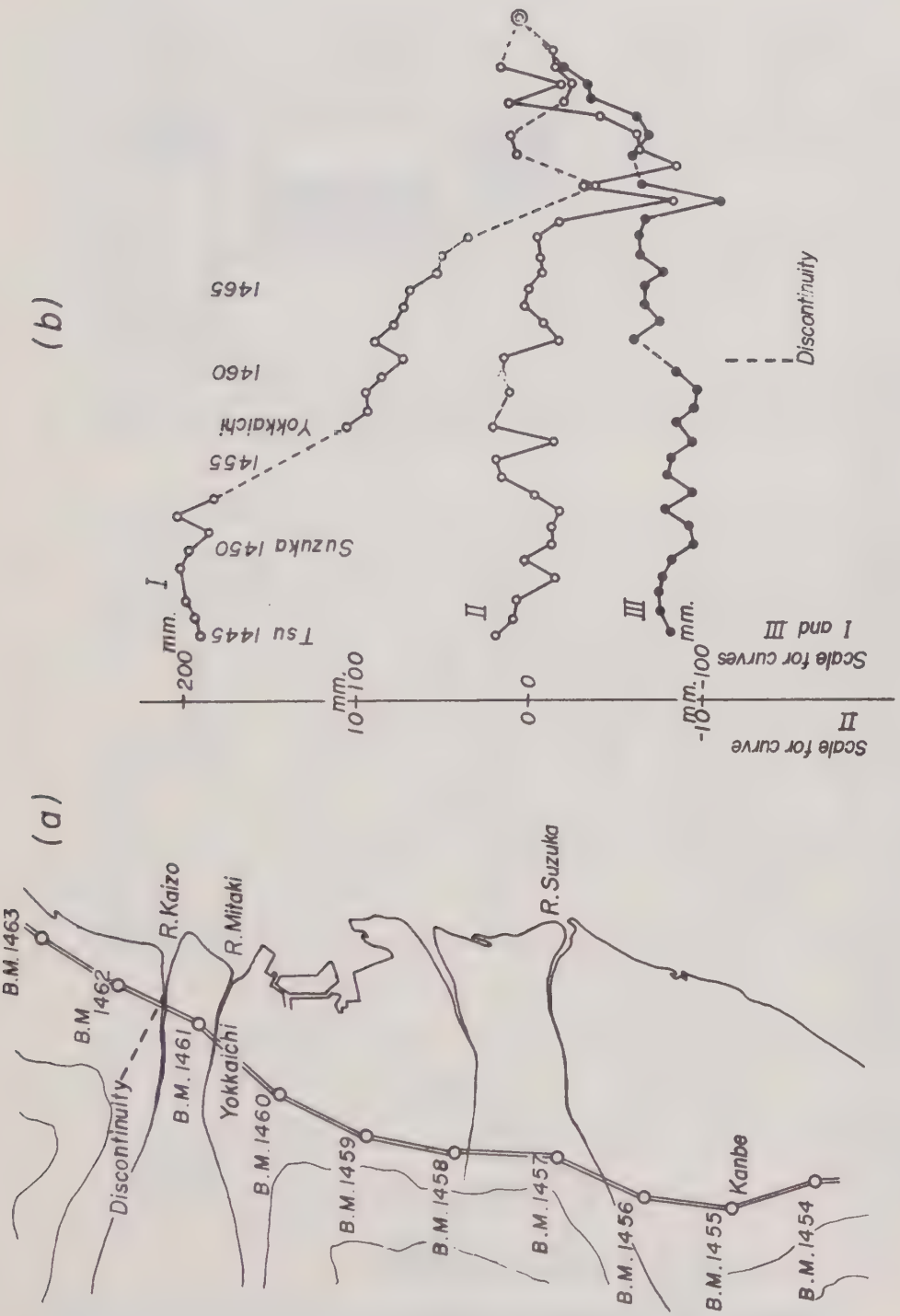

Fig. 2. (a) Leveling route.
(b) Tectonic changes.

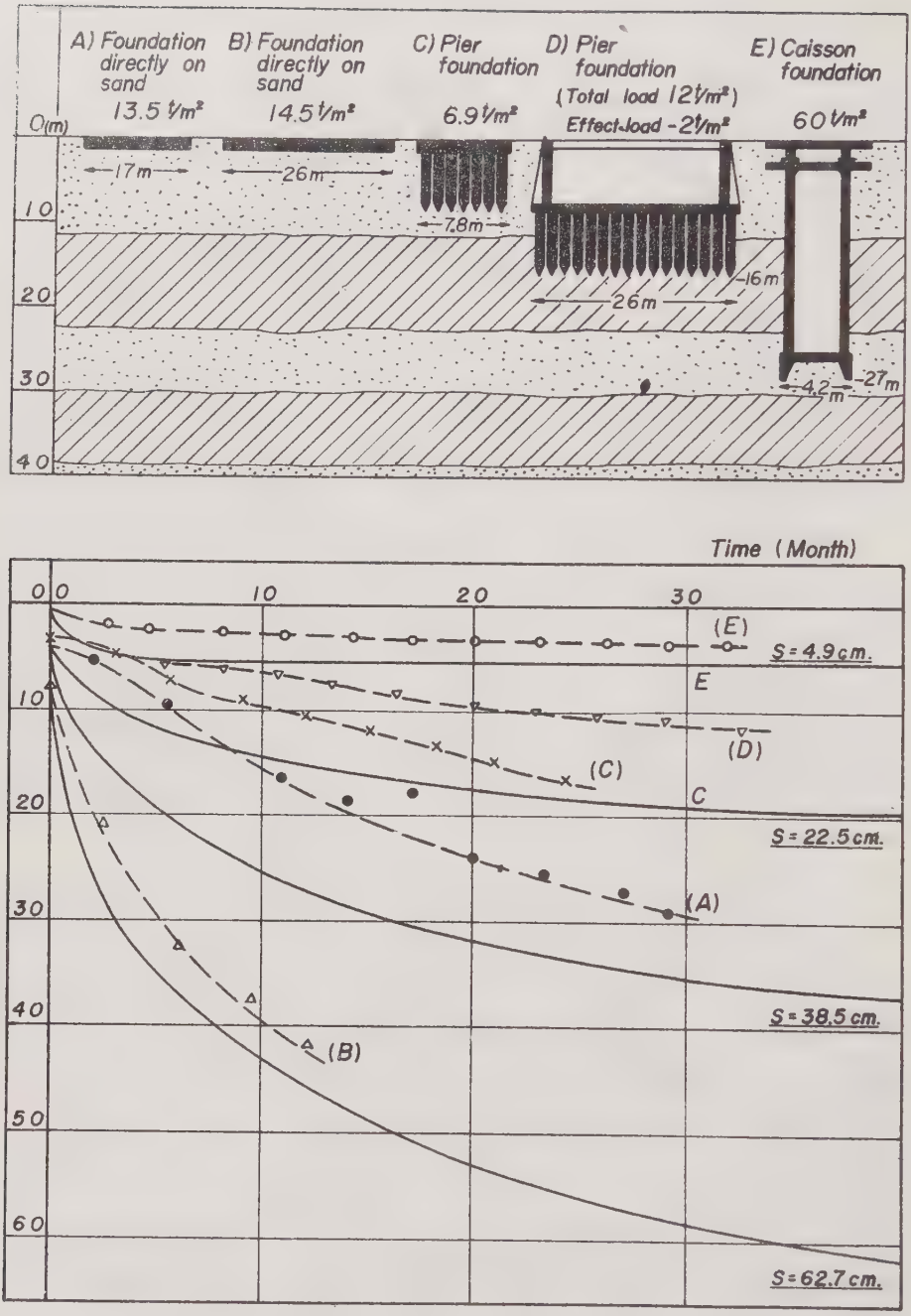

Fig. 3. Subsidence of various structures.

Curves in full-lines are calculated and those in broken-lines are actually observed.

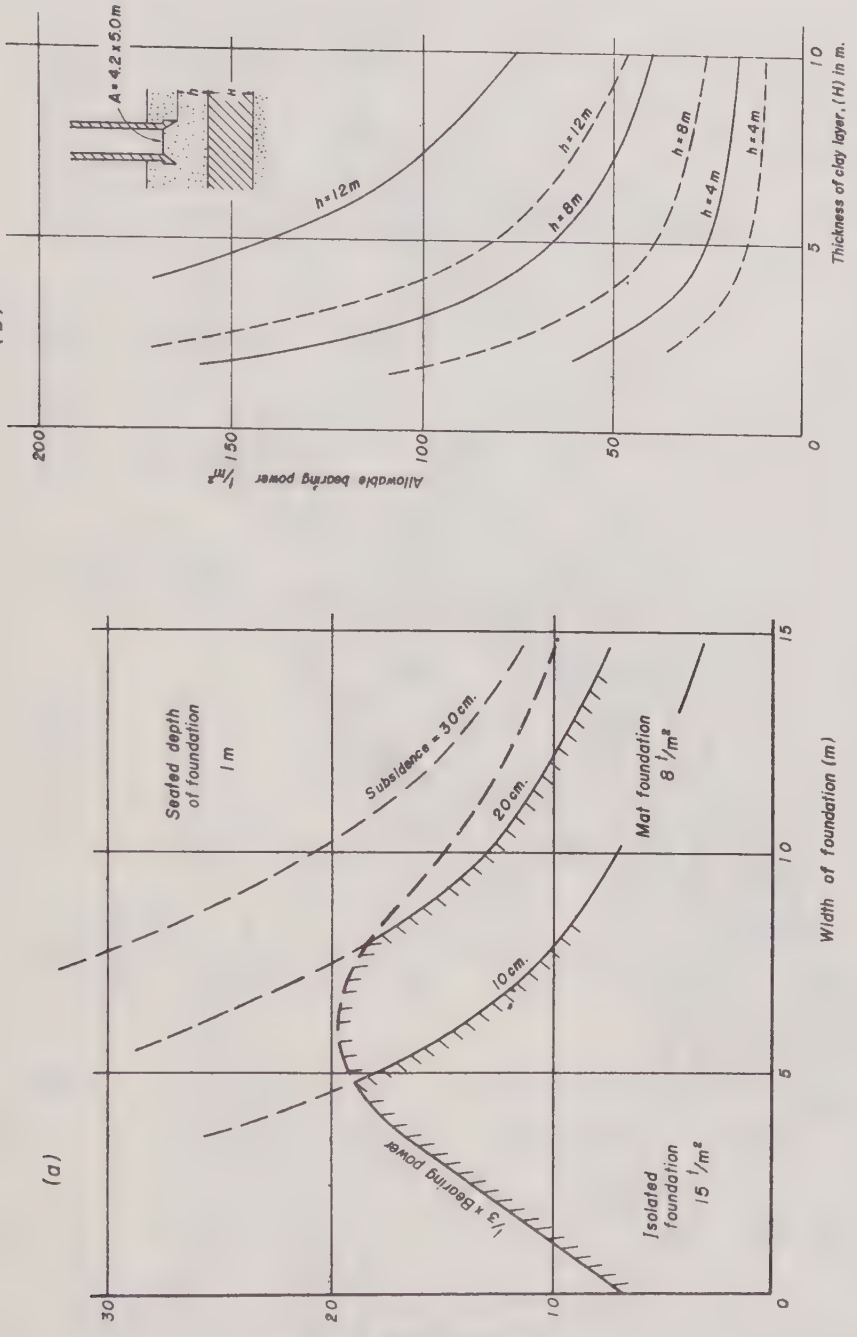

Fig. 4. (a) Allowable bearing power on the foundation supported by the upper sand layer 10m in thickness. Loads on the foundation which are responsible for the subsidence of 30, 20 and 10cm are calculated. Thick-line indicates the one-third of the maximum bearing power. Values of the bearing powers commonly used for the reinforced concrete buildings are shown according to the type of foundation.

(b) Load intensity on the pier of which base-area is 4.2 X 5.0m. Loads are responsible for the subsidence of 3 and 5cm. Calculation was made for various values of H and h.

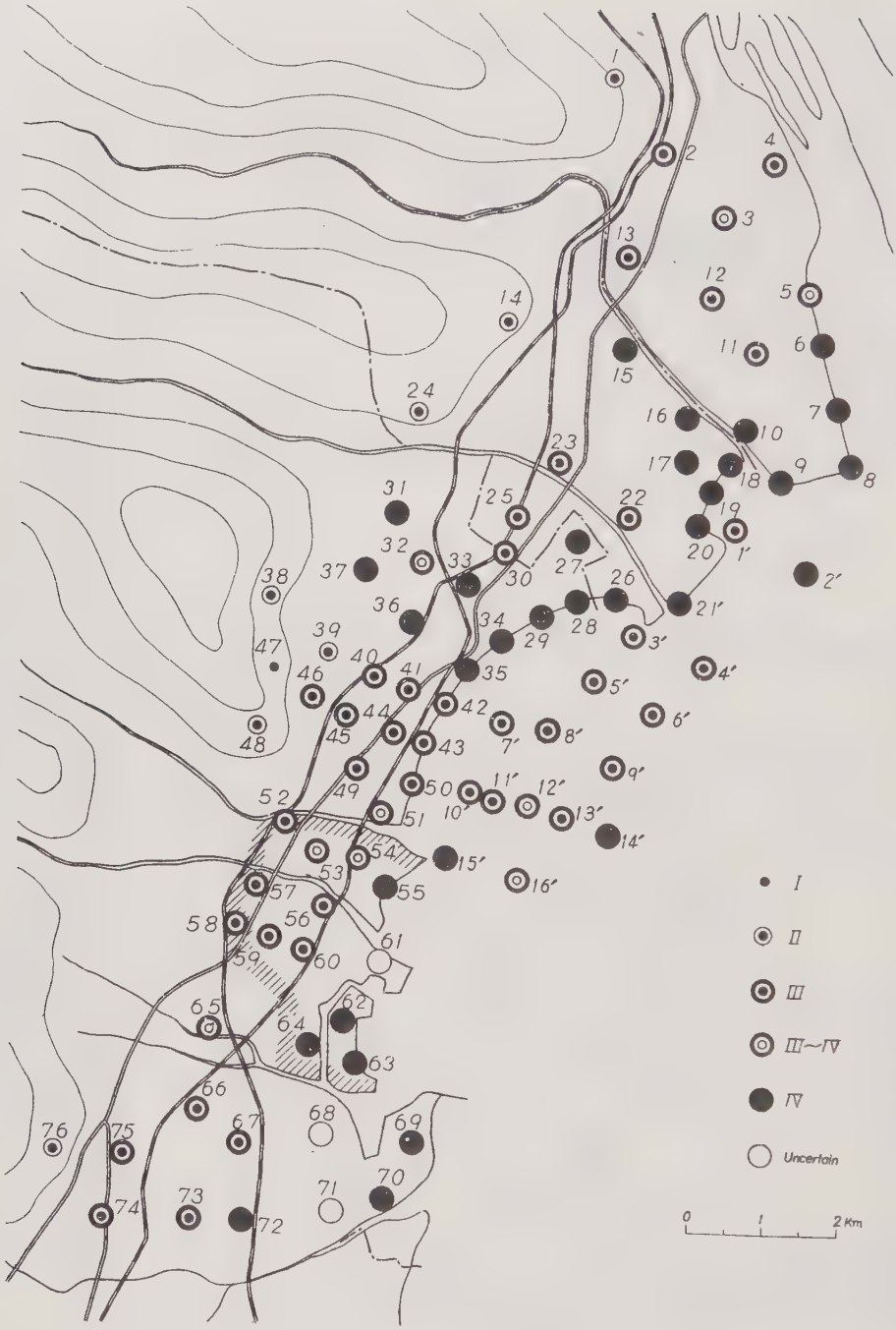

Fig. 5. Points of observation of the micro-tremors and the designated kinds of ground at these points.

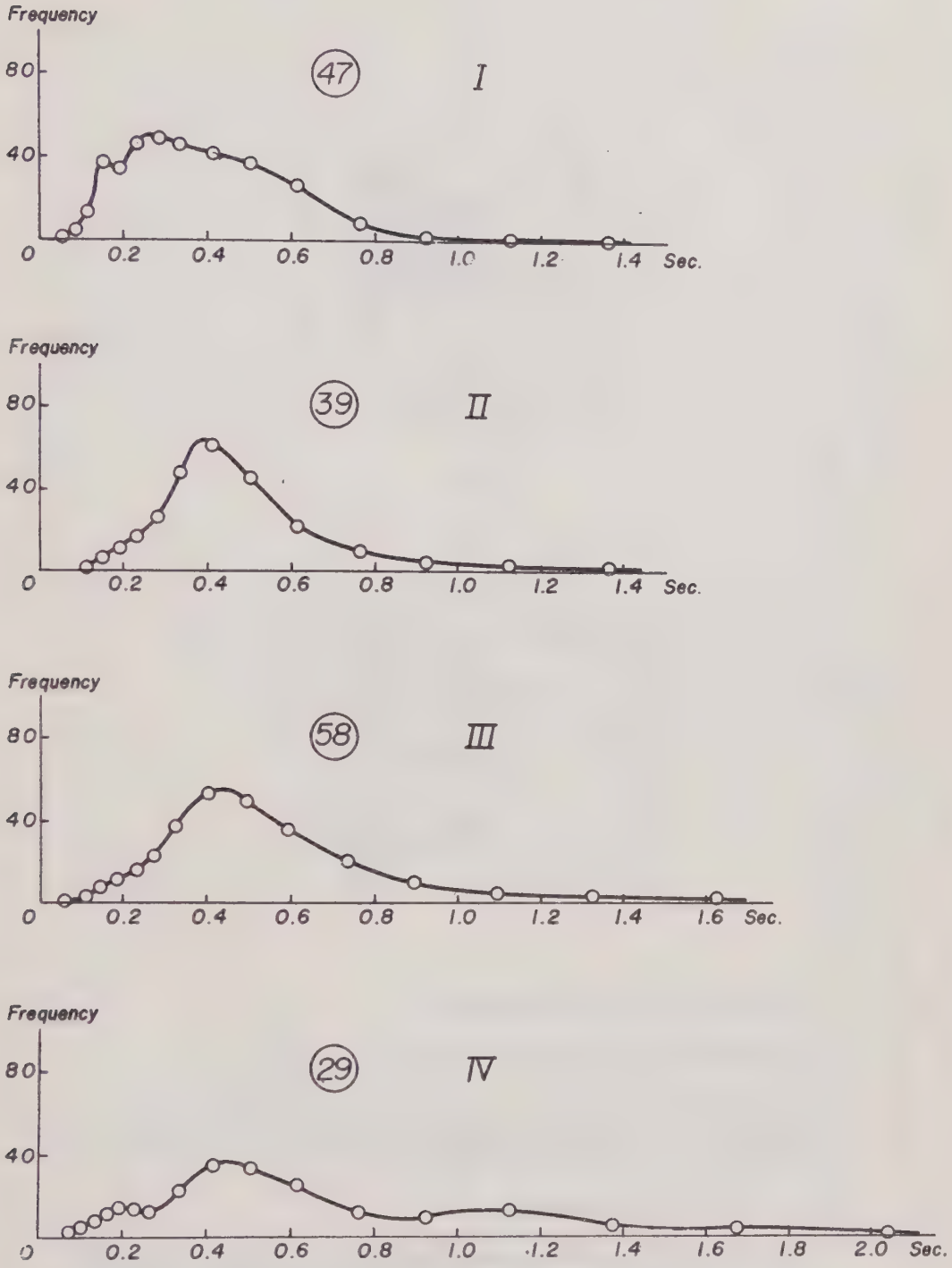

Fig. 6. Specimen Frequency-curves of micro-tremors.

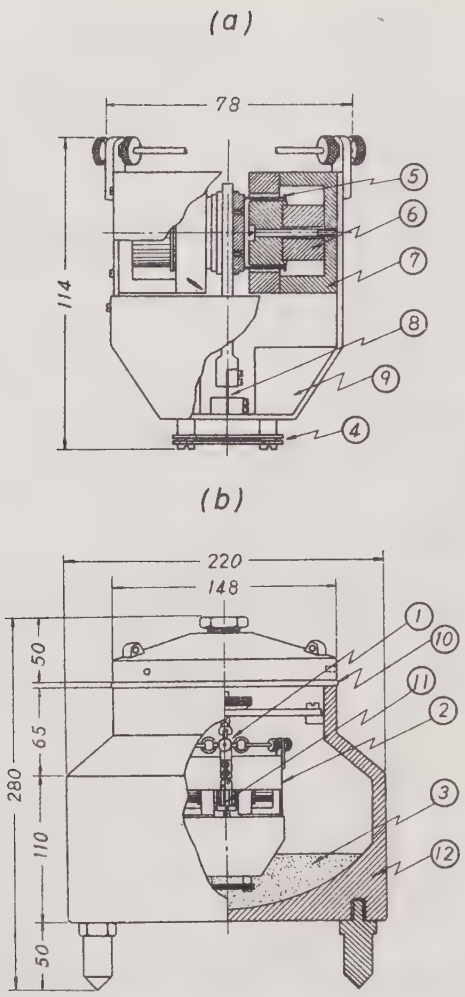

Fig. 7. Seismograph for sea-observation.

(a) Pendulum of seismograph and brass case.

4 .. electric heater, 5 .. moving coil, 6 .. core piece of magnet, 7 .. parmanent magnet, 8 .. supporting spring of pendulum, 9 .. additional weight.

(b) Water-proof container.

1 .. hanger, 2 .. brass case, 3 .. paraffin, 10 .. packing, 11 .. electric contact for detecting the vertical position of the pendulum, 12 .. container.

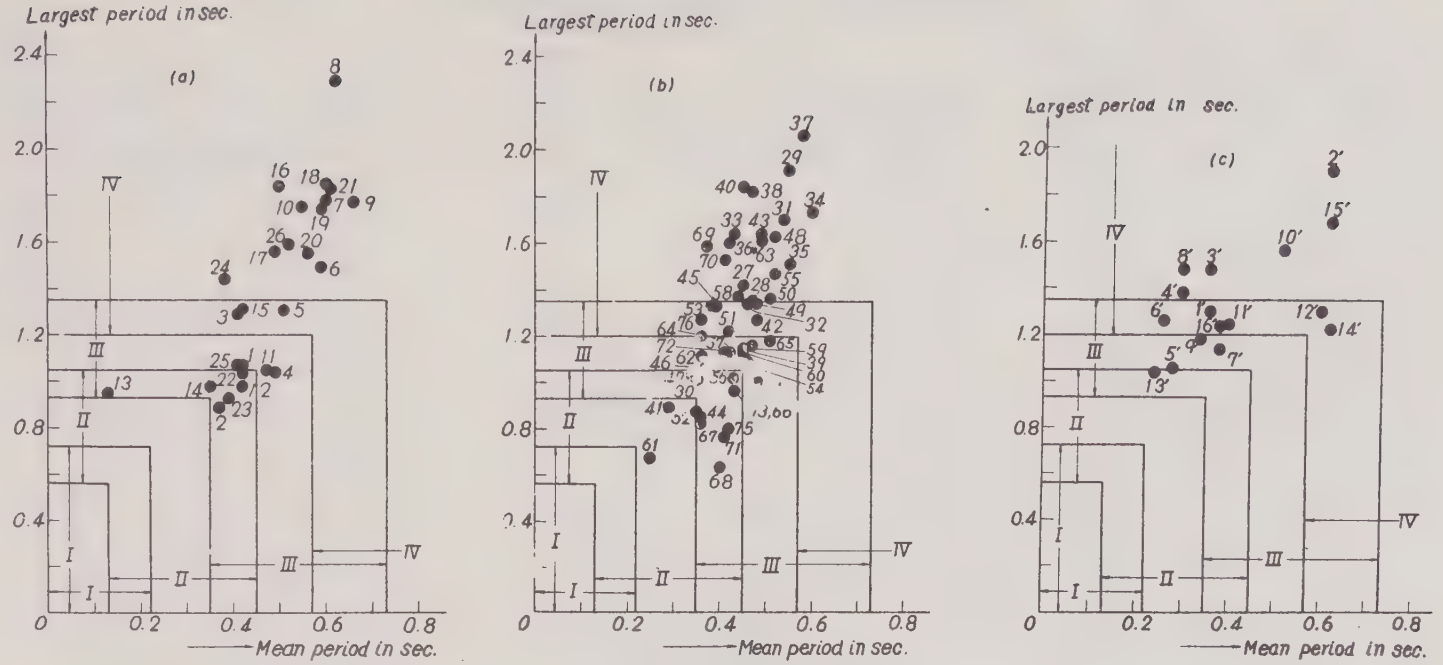

Fig. 8. Classification of ground (First proposal).

(a) On the land : Kuwana and Kawagoye areas.

(b) On the land : City of Yokkaichi.

(c) On the sea-bottom.

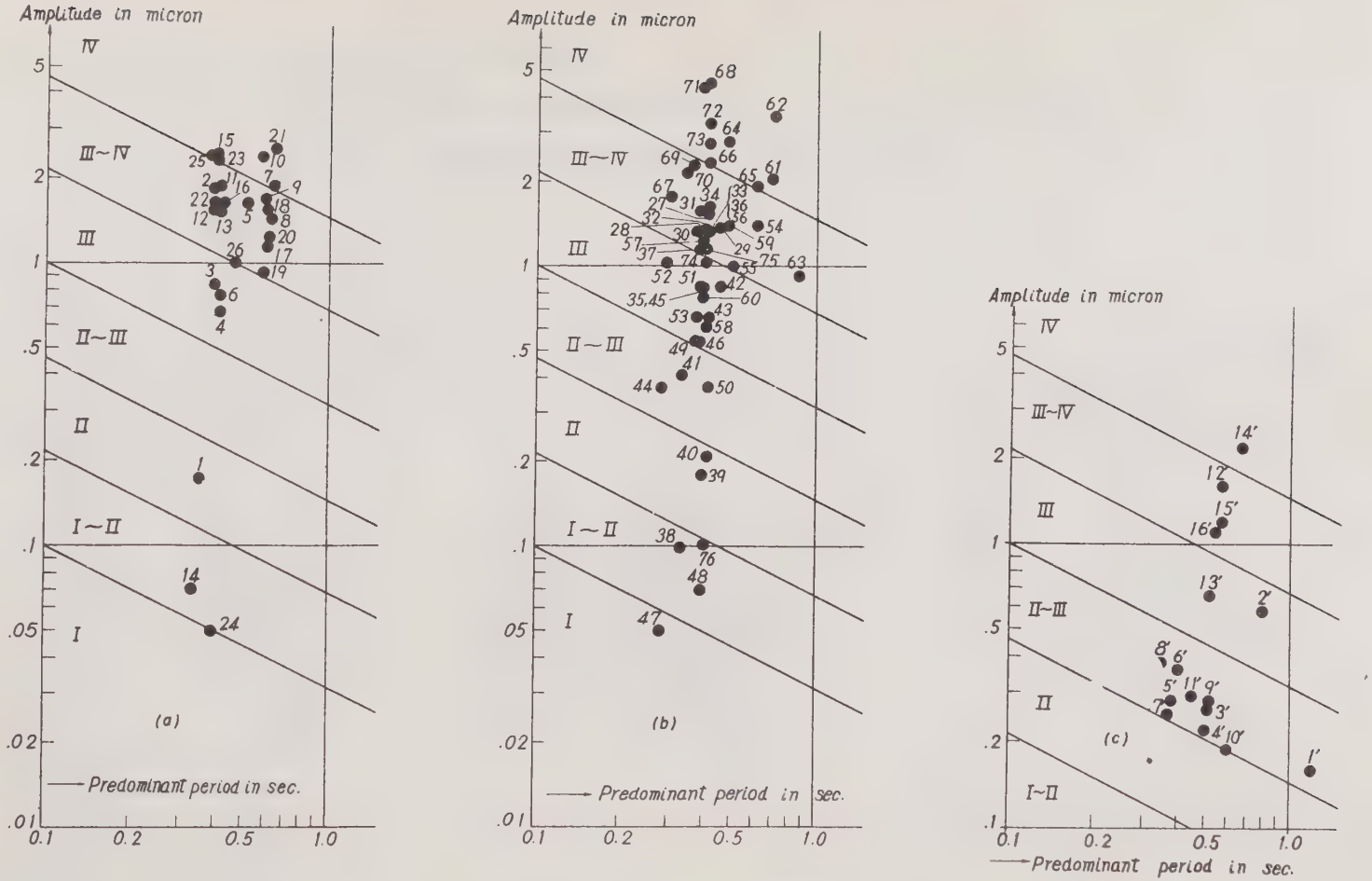

Fig. 9. Classification of ground (Second proposal).

(a) On the land : Kuwana and Kawagoye areas.

(b) On the land : City of Yokkaichi.

(c) On the sea-bottom.

DISCUSSION

V. A. Murphy, New Zealand Government Railways, New Zealand:

1. Please refer to the 3rd paragraph on the 7th page of your paper.
Kind III what do you mean by bluff formation?

2. Refer to the last paragraph on the 7th page of your paper.
How many points are necessary before classifying the kind?

3. Refer to the middle of page para. on the 8th page of your paper starting "In general ..... structure".
How do you define "diluvium"? Is it done by soil testing or by geological age?

N. Nasu:

1. The explanation is not good, I think.

2. One point for one place.

3. Not by the soil test, but by the geological consideration.

DAMAGE PRODUCED BY SMALL GROUND MOTIONS

By

Jack L. Alford*

INTRODUCTION

Motion of the ground results in vibration of structures. Such vibration is the same, in a qualitative sense, and is subject to the same physical laws, whether the source of the ground motion is natural (i.e., an earthquake) or man-made (i.e., from explosions, vehicular traffic, or the operation of heavy machinery). Quantitatively, the difference may be great since even the underground explosion of an atomic bomb involves an energy release into the ground which is equivalent to that of only a minor earthquake (1,2). Although the intensity of ground motion in the vicinity of man-made seismic disturbances is seldom great enough to cause loss of life or structural collapse, it may be sufficient to raise questions of property damage or nuisance. The frequency of such problems can be expected to increase in some proportion to the growth of urban population centers. It is of interest, therefore, to understand the effects of seismic disturbances in order that the possibilities of damage and nuisance can be balanced against the necessity for economical conduct of the disturbance-producing operations.

Most of the attention devoted to this problem has been concerned specifically with the effects of the use of commercial explosives. Few scientific investigations of the problem have been undertaken. Although many field observations have been made, these have seldom been comprehensive and the results are usually considered to be proprietary information. Nevertheless, impelled by economic pressure to use explosives at maximum effectiveness and limited by legal liability for the consequences of their actions, operating engineers have had to develop criteria for deciding what constitutes a "safe" seismic disturbance. The various criteria which have been developed may be classified in three groups, which are discussed in the following sections.

DIRECT CRITERION

The direct, or phenomenological, approach to the problem of blasting damage involves the correlation of blasting variables with reports of damage in neighboring structures for a number of blasts which is large enough to warrant generalization of the results. The important blasting variables are weight of explosive, distance from blast to neighboring structures, and geological conditions between the blast site and the structures. Damage reports, to be meaningful, should be based on careful inspections of the structures in question made before and after each blast by competent and unbiased observers. Several studies have been made which satisfy all or almost all of these requirements.

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Crandell (3) reports an investigation in which many structures were examined for evidence of damage caused by blasting operations. Although detailed data are not given for damage or for blasting conditions, the results lead to the threshold-of-damage relation shown in Figure 1. Each point on the line represents, for the given distance, a maximum charge of explosive above which damage is probable. The solid portion of the line indicates the range of blasting variables covered by the observations.

Damage to six buildings in a controlled series of blasting tests is documented by Edwards and Northwood (4). The threshold-of-damage line by means of which they describe their results is also shown in Figure 1.

For comparison, Figure 1 also includes points for two very large quarry blasts. In the first blast 373,000 pounds of explosive were fired approximately 1100 feet from a large mill building (5,6). In the second blast 1,347,000 pounds were fired approximately 1250 feet from the same mill building; complete information will be given in a forthcoming publication. The question of damage is difficult to assess since the mill building lacked the plaster which usually shows the first sign of minor damage. Even if it is assumed, however, that these two blasts represent conditions of incipient damage, it is apparent that many more data points are necessary to extend the existing damage criteria to such large charge weights.

By applying factors of safety to damage criteria, recommendations of maximum "safe" charge versus distance are derived. Figure 2 shows "safe" charge recommendations corresponding to the damage criteria of Figure 1. For comparison, Figure 2 also shows several other limit charge recommendations. Line 3 is the limit recommended by Langefors, Westerberg, and Kihlstrom (7); it is based on an extensive series of tests, all of which were fired in rock at distances of 100 feet or less. Line 2 is the limit proposed by Teichmann and Westwater (8) for clay soils; it is based on fewer instances of actual blasting damage than that of either Edwards and Northwood or Langefors et al. Line 5 represents the limit specified by the New Jersey Commissioner of Labor and Industry for blasting in "normal overburden." It is based on the field observations of the U. S. Bureau of Mines (9), in which only one instance of actual blasting damage was encountered.

Relations of the type shown in Figure 2 have the advantages of depending only on information which requires no instrumental observations, is known in advance of the blast, and is entirely within the control of the blaster. Such relations are widely used in practice because of their convenience. The lack of agreement among them is not surprising in view of the many detailed aspects of the blast and of the surrounding structures which are not taken into account. The limits recommended by Edwards and Northwood (for sand-clay and glacial till) and by Langefors et al (for rock) are not only the most conservative, but they have the most sound experimental basis. In the absence of local experience and where it is not feasible to make instrumental observations, they constitute the best available guide.

INDIRECT CRITERIA

Many attempts have been made to relate damage to some property of the ground motion produced by blasting. Such criteria depend upon measurements of the ground motion and are therefore not as convenient in practice as a simple weight-distance relation. They have the advantage that they do not depend upon the characteristics of the structures in the vicinity of the blasting, which may be too numerous for detailed consideration. Among the criteria which have been proposed are the maximum values of the displacement, the velocity, and the acceleration of the ground motion. Such criteria, like the direct criterion, depend for their validity upon the observations of actual blasting damage which support them.

It has been found (4, 8, 9) that the relation between damage and ground displacement or acceleration is dependent on the frequency of the ground motion, large displacements being tolerable at low frequencies and relatively large accelerations being tolerable at higher frequencies. This frequency dependence can be eliminated, to a good approximation, if comparisons are based on maximum ground velocity. Crandell's results (3), although expressed in terms of an "energy ratio," are equivalent to a critical ground velocity of 4.7 inches per second, above which damage is predicted. Edwards and Northwood (4) propose a critical velocity of 4.5 inches per second on the basis of their measurements. Damage was experienced above 110 millimeters per second (4.3 inches per second) in the work of Langefors et al (7). Such unanimity is remakable in view of the many uncertainties remaining in the problem.

Allowing a margin of safety, Edwards and Northwood recommend an operating limit on ground velocity of 2 inches per second. The New Jersey regulation requires that the allowable ground displacement be reduced with increasing frequency of ground vibration according to Table 1. It is interesting to note that this has the effect of limiting the maximum ground velocity (calculated on the assumption of simple harmonic motion) to 2 inches per second.

TABLE 1

FREQUENCY-AMPLITUDE RELATIONS

Frequency of ground motion, CPS	Maximum amplitude of ground motion, inches
Up to 10	Not more than 0.0305
20	0.0153
30	0.0102
40	0.0076
50	0.0061
60	0.0051

BUILDING RESPONSE CRITERION

The preceding sections indicate the extent to which investigators are agreed concerning the seismic effects of blasting operations. Despite recent contributions, however, a considerable degree of uncertainty remains and it is therefore necessary to use large factors of safety to control the risk of damage. Can this uncertainty be reduced, with a corresponding increase in the efficiency of blasting operations?

The accumulation of competent damage reports will lead, in time, to improvement of the existing criteria and many apparent conflicts will be resolved. Meanwhile, understanding of the damage problem could be increased by consideration of the relation between damage and the structural properties of buildings. Although it may be neither practicable nor desirable to control blasting by predicting the dynamic stresses in particular buildings, it is now feasible to analyze the response of typical structures to a wide variety of ground motions. This process of analysis, called the response spectrum technique, has been described by Hudson (10). By performing many such analyses and interpreting the results in a statistical manner, as has been done in the similar problem of earthquake damage (11,12), it will be possible to draw general conclusions concerning the structural effects of blasting. This knowledge, in turn, would permit more exact interpretation of blasting damage observations.

Consider, for example, the response spectrum shown in Figure 3; it represents the response of a series of idealized structures to the ground acceleration which was recorded in the basement of a large mill building at the Corona, California quarry blast of July 1952 (5). Using this spectrum and the fundamental vibration period of the building, calculated from structural plans, it was possible to predict the vibration of the building which was measured independently at the time of the blast (6). This example demonstrates that, if the ground acceleration caused by a blast is known, the response spectrum is a significant means for summarizing the structural effects of the blast. It also shows that the response spectrum is more directly useful than the recorded ground motion itself.

The importance of such information to the further evolution of damage criteria can be demonstrated by means of a hypothetical example. Suppose that two mill buildings of similar design and construction, instead of one, had been subjected to the ground motion of the 1952 Corona quarry blast. Suppose also that one of these buildings, like the actual structure, had a fundamental vibration period of 0.35 second and that the second building had a fundamental period of 0.25 second. Examination of Figure 3 shows that the vibration response of the second building (and hence the dynamic stresses) would have been from two to four times as great as that of the first building, depending upon the degree of structural damping appropriate to the design. Under these circumstances it is probable that one building would be damaged and that the other would not. It is clear that a damage criterion based on the performance of the longer-period building would be hazardous for the shorter-period building. Conversely, a damage criterion based on the performance of the shorter-period building might be unnecessarily conservative for buildings of longer period.

Similar conclusions result if one considers the behavior of two other hypothetical structures, both having a fundamental period of 0.25 second but differing markedly in their damping properties. An all-welded, steel structure might have a fraction of critical damping as low as 0.02; whereas a reinforced concrete building with hollow-tile filler walls would have approximately 0.15 of critical damping (13). Again it is clear that the differences in structural properties would significantly influence the interpretation of any resulting damage.

RECOMMENDATIONS FOR FURTHER INVESTIGATION

For the reasons outlined in the preceding paragraphs, it is desirable that future investigation of the blasting damage problem include the collection of information on the shape and magnitude of response spectra as related to weight of explosive charge, distance, and type of ground. To this end it will be necessary that measurements of ground acceleration be added to the instrumental observations now commonly made. Acceleration measurements might very well replace measurements of displacement and velocity since ground velocity can be determined by integration of the acceleration record and since displacement is not required in order to determine structural effects.

The usefulness of such information will be increased if the work of various investigators has some common basis. For this purpose the following recommendations are offered.

(a) Blasting data. Complete information is needed on number of holes fired, type and quantity of explosive per hole, and whether breakage is free or confined. If delays are used, it should be possible to determine the actual charge fire at each instant.

(b) Distance and type of ground. Distance from blast point to measurement point should be measured, and presented by means of a map showing the relation between blast and neighboring structures. Description of soil conditions should be given in standard geological terms and supported by geological data.

(c) Damage observations. Nearby structures should be inspected in detail before and after the blast, by persons qualified in structural mechanics. Existing damage should be marked and recorded, by photographic means if possible, for objective comparison with the results of post-blast inspection. A general description of each structure should be given, together with any available information on the dynamic properties of the structures.

(d) Instrumentation. Complete acceleration-time histories of three components of ground motion should be recorded: one vertical and two horizontal at right angles to each other. When measuring on free ground, one horizontal component should be oriented toward the blast. When measuring at the base of a structure, the horizontal components should be aligned with the principal axes of the structure. Care should be taken that the accelero-

meters are tightly coupled to the ground or foundation in order that true ground acceleration be sensed. The accelerometers and the recording system should be free from dynamic errors in the range from zero to 100 cycles per second. Instrument characteristics and calibration data should be included in the results.

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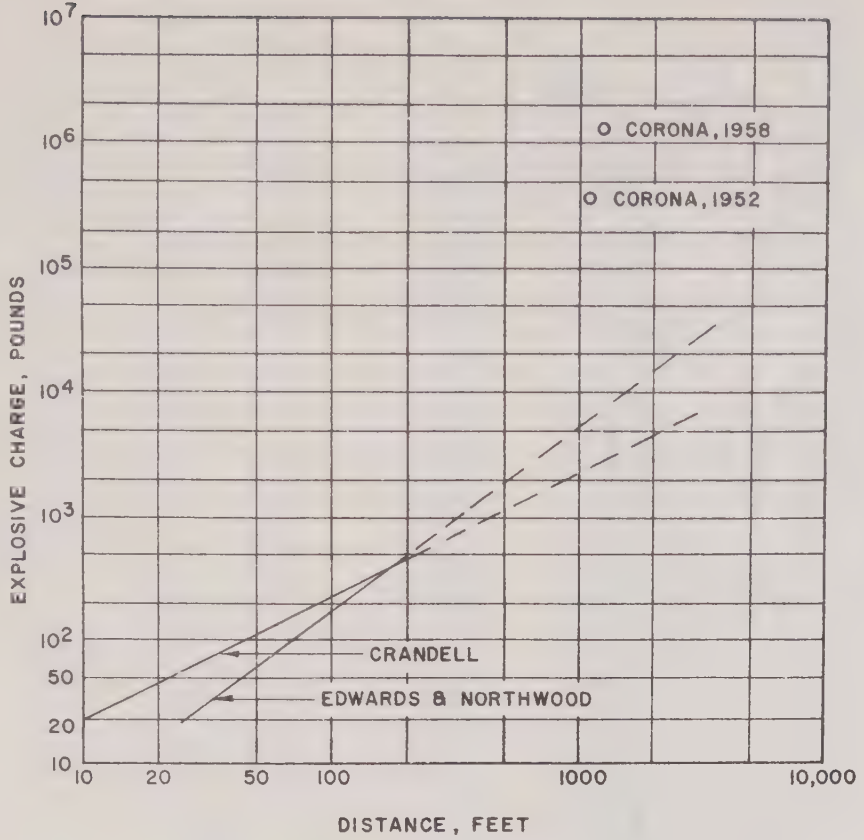

Figure 1. Comparison of predicted damage lines with data from two large quarry blasts.

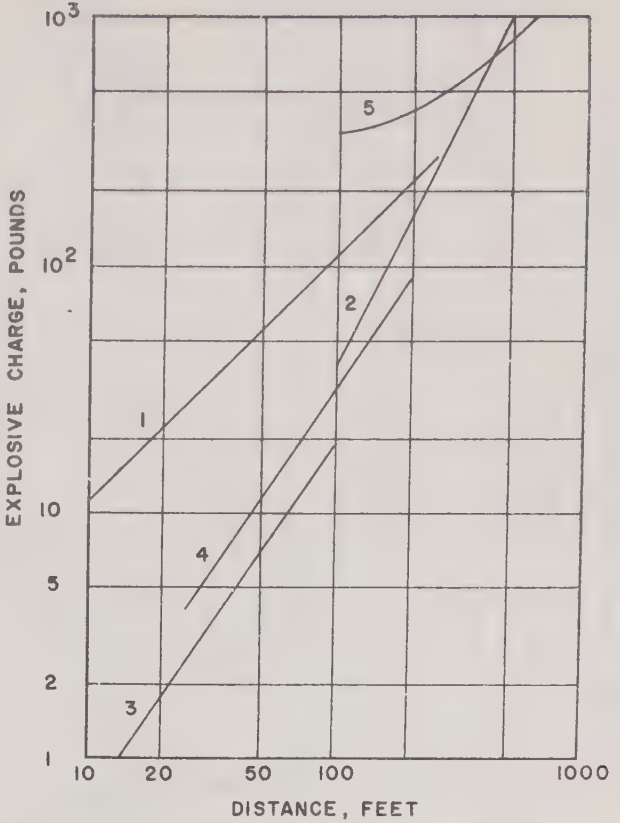

Figure 2. Comparison of limiting charge versus distance relations recommended by: 1) Crandell; 2) Teichmann & Westwater (clay); 3) Langefors et al. (rock); 4) Edwards & Northwood; 5) New Jersey regulation.

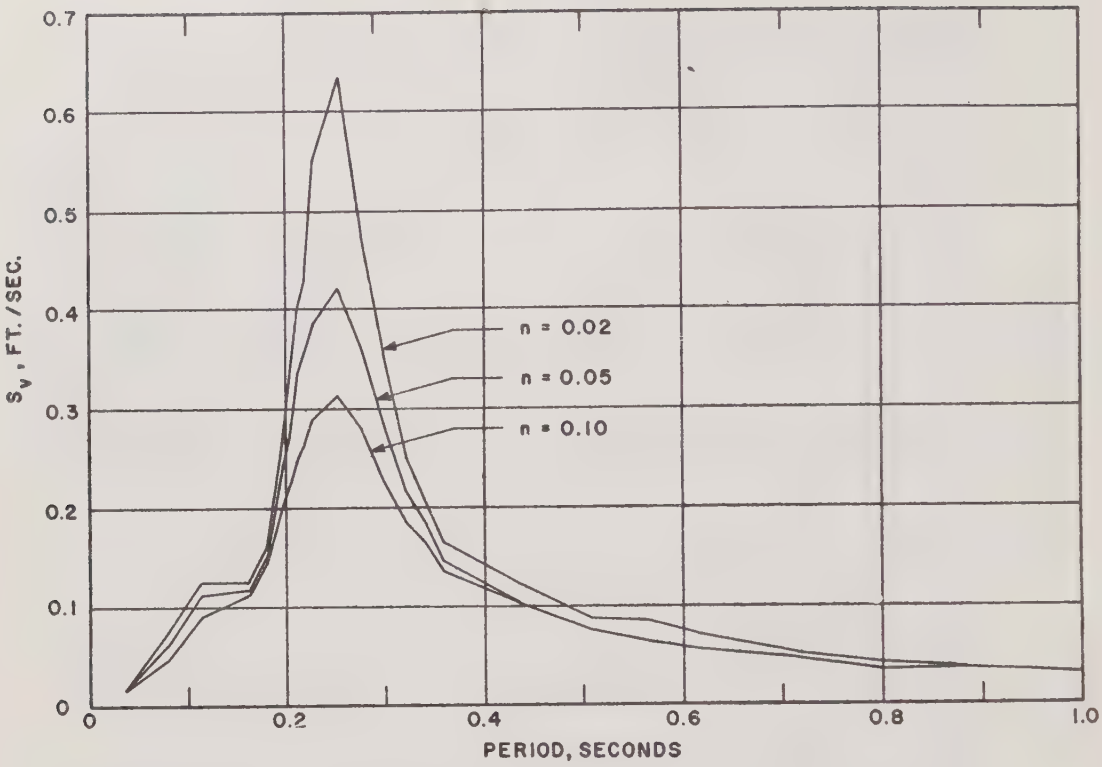

Figure 3. Velocity response spectrum for horizontal component, N31W, Corona, California, 26 July 1952. Charge weight: 373,000 pounds. Distance: 1100 feet.

DISCUSSION

F. W. Evans, British General Electric Co. of Japan Ltd., Japan:

I was very interested to see that Prof. Alford makes reference to the work of Teichman and Westwater and that he shows their curve for the limiting explosure charge and distance.

On a large unclear power station site in Scotland I had occasion to make use of the figures and formulae they give in their paper.

We had to satisfy the Atomic Energy Authority that reacter foundations under construction would not be subjected to vibration amplitudes above a certain value, by adjacent blasting for other works. By using their formula, the factors (to determine the probable amplitude of vibration) are:-

The weight of explosure charge
The distance
And a constant which depends on the
ground nature (in this case rock).

We were able to lay down limiting weights of explosure charge to be used at various distances from the structures to be safeguarded.

Smaller charges than those permitted were first fired and the amplitudes of vibration they produced were checked with a vibragraph. The results agreed quite closely with those derived from the formula.

J. L. Alford:

I thanked Mr. Evans for his comment and asked if it were possible to obtain the data for the blasts to which he referred.

The Seismic Intensity at a Certain Dam Site

By IKUEI MURAMATU* TOKUTARO YABASHI**

1 Introduction and some remarks

The vibration of any system due to an earthquake may be discussed conveniently by means of spectra. Let B(T) be the spectrum of incident earthquake wave at the boundary of bed rock, and G(T) and S(T) be the vibrational characteristics of the overlying ground, and the structure, respectively. Then spectrum of actual earthquake motion at the ground surface is expressed by B(T) x G(T), while the spectral response of a structure on the ground surface in an earthquake may be obtained by B(T)x G(T) x S(T).

Therefore, for the design of earthquake-proof construction we must have recourse to each of the spectra, above mentioned. It is now well recognized that the structure is liable to resonate with the predominant motion of the ground so that the coincidence of the two natural periods of the structure and the ground has a serious effect on the structure. Moreover, recently the general characteristics of grounds have been investigated, especially by K. Kanai, and empirical formulas for these spectral responses are derived. But the earthquakes studied in the derivation of his formulas are almost limited to weak ones. Therefore, there still remains some doubt whether the formulas are applicable to destructive earthquakes or not. Our main effort in the present study has been devoted to the clarification of this question.

Another important factor for the design of an earthquake-proof structure is to estimate the maximum seismic intensity to be expected at its construction site in the life time of the structure. As our knowledge is still too meagre to guarantee the prediction of the earthquake occurrence, we are obliged to discuss the activity in the future on the statistical point of view based on the data of past seismic activity.

With this view in mind, we estimated the maximum seismic intensity to be expected at least once in 100 years at Yokoyama, Gifu Prefecture, where a dam is being constructed. We also made some efforts to study vibrational characteristics of the ground at that dam site.

2 Comparison of the seismic intensity

Observations of seismic intensities in this area under the Meteorological Agency have been made by an observer living at Higashiyokoyama Power Station, located at a distance less than 1000 meters from Yokoyama Dam Site. We compared the seismic intensity at this point with that observed at the Gifu Meteorological Station. The former is located in a mountainous district and the latter is situated on the Plain. They are 34km apart.

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The result of this comparison is shown in Table 1.* For destructive earthquakes, the seismic intensity at Higashiyokoyama is about 1/4 times that at Gifu, while, on the contrary, for weak earthquakes the former is about 1.7 times of the latter.

3 Acceleration spectrum of an earthquake motion

Empirical formulas to estimate the acceleration spectrum of an earthquake motion are given by K. Kanai as follows 4) :

$$\alpha(T)_{T_0} = B(T) \cdot G(T)_{T_0} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (1)$$

where $\alpha(T)_{T_0}$, $B(T)$ and $G(T)_{T_0}$ are the acceleration spectrum of an earthquake motion, the spectrum of the earthquake waves arriving at the bed rock, and the vibrational characteristics of the ground overlying the bed rock, respectively. Suffix T_0 shows the predominant period of the ground while T denotes an arbitrary period to which we are concerned.

First, Kanai expressed $B(T)$ as follows:

$$B(T) = C(M,\Delta)\, T^{-1} , \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (2)$$

for $T \leqq Tm$, which is empirically given by

$$\log Tm = 0.39\, M - 1.70 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (2').$$

The meaning of the equation (2) is that the velocity spectrum of the earthquake wave arriving at the bed rock is constant upto the maximum period Tm corresponding to the maximum value of the displacement spectrum of the earthquake wave. In the above equations M shows the magnitude of the earthquake while Δ is the distance from the epicenter to the observing point.

Next $G(T)_{T_0}$ will be explained. Theoretically speaking, as Sezawa and Kanai have shown, there exist many high harmonics in the vibration of the stratified ground. Kanai took only the fundamental mode into consideration, and simulated the vibrational characteristics of the ground by that of a displacement seismometer. This assumption well conforms with the observational fact for motions having periods near the predominant period of the ground. But if we want to apply the formula for the motions of longer period, Kanai's formula for $G(T)_{T_0}$ coverges to zero and does not tend to a finite theoretical value. This inconsistency is revised if we introduce a factor u^2 into his formula, and put

$$G(T)_{T_0} = 4(1+\beta)^{-1}\, u^2 \left\{ (u^2-1)^2 + (\tau u)^2 \right\}^{-\frac{1}{2}} \quad \cdots\cdots \quad (3)$$

---

* In Table 1, the seismic intensity I is connected to the acceleration by the following relation $\alpha = 0.45 \times 10^{0.5I}$ (by H. Kawasumi).

In this comparison, the correction of the seismic intensity by the epicentral distance is made as follows: Since we are eventually dealing with the damage brought by earthquakes, we examined $\alpha - \Delta$ relation in $\Delta < 200$ km for many earthquakes, and found the relation $\alpha \propto \Delta^{-1 \sim -2}$. However, we could find no appreciable difference between the mean values of their intensity ratios for several available earthquakes, worked out for two cases.

By this revision, the formula obtained does not mark any appreciable difference from Kanai's formula in the vicinity of u = 1, around which his formula is intended to be used. And the range of applicability of $G(T)_{T_0}$ is much extended to the larger values of T. In this formula β denotes the ratio of mechanical impedance of the overburden to that of the bed rock. According to Kanai's study β is nearly constant regardless of the kinds of grounds.

$$\beta = 1/5 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3')$$

The damping constant in terms of critical damping $(1/2)\tau$ is also given by Kanai by the following formula

$$\tau = 0.2\sqrt{T_0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3'')$$

for the grounds of various kinds.

By these equations, we can calculate the acceleration spectra of an earthquake on various grounds. The results are shown in Fig. 1.

4 Vibrational Characteristics of the Ground at Yokoyama Dam Site and at Gifu Meteorological Observatory.

We carried out seismic observations by means of high sensitive electromagnetic seismographs and an ordinary seismograph at various points in Yokoyama Dam Site.

The observations by high sensitive seismographs were made for 3 months only at night, but 335 small earthquakes could be recorded. The tremors caused by an explosion of 150 ton dynamite at Miboro were also recorded at various points near the Dam Site simultaneously with fast paper speed of recording. In calm times, micro-tremors in a horizontal hole made in the Dam Site were recorded too, with a super-high sensitivity of about 10 mm/kine. In the routine observation by means of an ordinary seismograph with a starter and a stopper we could catch 4 felt earthquakes.

The dominant periods obtained by analyses of these seismograms are shown in Tables 2 and 3. It was recognized that many dominant periods exist, but the period near 0.15 second is the most predominant.

The predominant period of the ground at Gifu Meteorological Observatory was searched from the seismograms of past destructive earthquakes, recent weak earthquakes, and micro-tremors. The results are shown in Table 4. The predominant period to be noted seems to be 0.4 - 0.5 second.

5 Interpretation of the Result of § 2.

In § 2, we found that the seismic intensities at Yokoyama are smaller than those at Gifu for destructive earthquakes, but for weak earthquakes, on the contrary, the former are larger than the latter. This phenomenon must be examined quantitatively by the use of equations mentioned in § 3 using predominant periods T_0 at Yokoyama and at Gifu, and paying due considera-

tion for the value of T.

The period T is to correspond to the maximum acceleration observed actually at each place for each earthquake.

Let us first examine the results for destructive earthquakes.

The determination of seismic intensities at Gifu and Yokoyama must have been made mainly from the response of ordinary wooden houses, because the intensities were larger than III at both points. As Japanese wooden houses are usually most sensitive to the vibrational force having a period near 0.5 second, the period of the motion taken up for the estimation of the seismic intensity must have been about 0.5 second. Then the seismic intensity observed at Yokoyama is calculated as

$$\alpha_Y^{(d)}(0.5) = B(0.5) \cdot G(0.5)_{0.15} \quad \cdots\cdots\cdots\cdots\cdots\cdots (4)$$

and that at Gifu is calculated as

$$\alpha_G^{(d)}(0.5) = B(0.5) \cdot G(0.5)_{0.5} \quad \cdots\cdots\cdots\cdots\cdots\cdots (4')$$

Let us next see the result for weak earthquakes. As the seismic intensities were I～II at both places, the determination of the intensities must have been made by human senses. The human senses have an almost uniform sensitivity for the vibration with a period above 0.1 second as reported by M. Ishimoto. Therefore, we may consider that both of the seismic intensities observed at Yokoyama and Gifu were the true maximum values of the ground motions, consequently the periods to be taken in our calculation may be put equal to the predominant periods T_0 at both sites. Then, the seismic intensity at Yokoyama is to be calculated by

$$\alpha_Y^{(w)}(0.15) = B(0.15) \cdot G(0.15)_{0.15} \quad \cdots\cdots\cdots\cdots\cdots\cdots (5)$$

and that at Gifu is similarly

$$\alpha_G^{(w)}(0.5) = B(0.5) \cdot G(0.5)_{0.5} \quad \cdots\cdots\cdots\cdots\cdots\cdots (5')$$

Consequently, the ratio of seismic intensities at Yokoyama and Gifu is expressed by (4)/(4') for destructive earthquakes; and by (5)/(5') for weak earthquakes. In performing these calculations by equations (2) and (3), we can omit $C(M,\Delta)$, for M and Δ are common. Magnitudes of all earthquakes used here are larger than 4, consequently Tm's are all larger than 0.725 second by eq. (2"). As the periods T dealt with here are smaller than this value, eq. (2) is applicable. Then we obtain the ratios as follows.

$$\alpha_Y^{(d)}/\alpha_G^{(d)} = B(0.5) \cdot G(0.5)_{0.15} / B(0.5) \cdot G(0.5)_{0.5}$$

$$= \frac{0.5 \times 3.3}{0.5 \times 12.5} = 0.264 \quad \cdots\cdots\cdots\cdots\cdots\cdots (4")$$

and

$$\alpha_r^{(w)}/\alpha_q^{(w)} = B(0.15)\cdot G(0.15)_{0.15} / B(0.5)\cdot G(0.5)_{0.5}$$

$$= \frac{0.5 \times 6.4}{0.15 \times 12.5} = 1.7 \qquad \cdots\cdots\cdots (5'')$$

These calculations may also be carried out directly on Fig. 1.

We have thus been able to explain the phenomena described in § 2 not only qualitatively, but also quantitatively by using equations (2) and (3) of the acceleration spectrum mentioned in § 3. We could see at the same time that the smaller seismic intensities at mountainous area in destructive earthquakes do not necessarily show that the motions in the mountainous area were actually smaller than those on alluvial plain. But they show that the motions of structures in the former area may be smaller than those of the same kind of structures on the alluvial plain, provided the natural periods of the structures are longer than the predominant period of the ground at the former site. Therefore, we can see also that in the mountainous area accelerations of long period vibrations are small but those of the short period vibrations may become remarkably large. All these mentioned above are shown in Fig. 1. Such a good agreement of the observational result with the present theory shows that the applicable range of Kanai's formulas may be extended to destructive earthquakes if it is slightly modified as above mentioned. The characteristics of destructive earthquake motion will be studied in detail again in § 8.

6. Activity of past earthquakes

H. Kawasumi has investigated the earthquakes in historical times and obtained the expectancy of the maximum intensity throughout our country. For instance, the expectancy for 100 years is shown in Fig. 2. But the values of the seismic intensity are determined under the assumption that the ground throughout the country has a uniform vibrational characteristics.

For Yokoyama Dam Site particularly, the expectancy of the seismic intensity estimated from the earthquakes occurred in the neighbourhood in historical times is obtained as follows.

If the number of earthquakes with intensities exceeding I at Yokoyama is expressed by S(I), the statistics show that

$$S(5) = 20, \quad S(6) = 11, \quad S(7) = 7, \text{ in past 1300 years,}$$

where 5, 6 and 7 means the strong (80 ~ 250 gal), the disastrous (250 ~ 800 gal), and the ruinous (800 gal <), respectively.

The results obtained from the activity of earthquakes during these 72 years since the beginning of the Meiji era (1868) are

$$S(5) = 16, \quad S(6) = 6, \quad S(7) = 3.$$

These numbers are extremely large as compared with those in historical times. This is due to the fact that the cycle of seismic activity is

very long (over 800 years), and, new phase of high activity set in recently with the Nobi earthquake of 1891 and the activity following it in this region were extremely high. Since the seismic activity in this region seems to have fallen to rest at present we may regard the activity of earthquakes in historical times (1300 years) as a standard activity in this region. Then the maximum seismic intensity to be expected to occur at least once in 100 years at Yokoyama area on the fictitious standard ground comes out to be about 200 gals, from the condition S(I) = 1300/100.

7. Estimation of the Expectable Seismic Intensity at Yokoyama Dam Site with Reference to the Vibrational Characteristics of the Ground

Important ground characteristics, from engineering point of view, consist of the vibrational characteristics and the fracture strength.

As rocky grounds are hardly broken, we have only to give attention to the vibrational characteristics. Now we are to estimate the expectable seismic intensity at Yokoyama Dam Site with due consideration of the vibrational characteristics of the ground. That is

$$\alpha_M(T_0) = B_M(T_0) \cdot G(T_0)_{T_0} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots (6),$$

where suffix M is written to show the maximum earthquake motion to be expected at Yokoyama within coming 100 years. On the other hand the expectable value on a standard ground in this district is expressed as follows.

$$\alpha_{M,S}(T_0,S) = B_M(T_0,S)\cdot G(T_0,S)_{T_{0S}} = 200 \text{ gals} \quad \cdots\cdots (7),$$

where suffix S means "on a standard ground*".

The reason we put T equal to $T_{0,S}$ in eq. (7) will now be explained. If the estimation of seismic intensity at every point was made from the data on vibrations or damage of Japanese wooden houses, it may be appropriate to take T in eq. (7) equal to the proper period of the usual wooden houses. However, the observations of seismic intensities used by Kawasumi were made by means of an intensity scale based on damage of various structures, deformation of the earth's surface and other items, and corresponding maximum accelerations were calculated by the intensity $\sim$ maximum acceleration relation determined from the observations by acceleration - seismographs. The period of the vibration which gave the maximum acceleration must therefore be the predominant period of the ground.

Then the expectable maximum value is calculated as follows,

$$\alpha_M(0.15) = \alpha_{M,S} \times B_M(0.15)\cdot G(0.15)_{0.15} / B_M(0.35)\cdot G(0.35)_{0.35}$$

$$= 200 \text{ gals} \times 0.35 \times 1.9/0.15 \times 3.0 = 306 \text{ gals} \quad \cdots\cdots (8)$$

---

* It is assumed here that on the standard ground the predominant period is 0.35 sec.

8. Seismograms of Great Earthquakes and the Relation $B(T) \propto T^{-1}$

The value of the acceleration given in eq. (8) seems to be larger than that used commonly in practice. The reason depends on the relation $B(T) \propto T^{-1}$, principally. When the relation was derived by Kanai, the basic data were restricted to seismograms of weak Japanese earthquakes recorded at the depth of 300 meters at a mine, although the relation obtained is concordant with the nature of strong earthquakes observed in U.S.A. Consequently, it may still be doubted whether the relation is applicable for destructive Japanese earthquakes or not. But we can not at present expect the seismograms of destructive earthquakes at deep subterranian points or on rocky grounds in mountainous area can be obtained in near future. Therefore, we examined the seismograms of recent destructive earthquakes obtained at Gifu Meteorological observatory with special attention to the short period components in the records. We could also examine the original seismograms of the Nobi earthquake of 1891 at the Gifu and Nagoya Meteorological Observatories. 1) 2) They are shown in Fig. 3.

The accelerations of the vibrations with long periods and large amplitudes and ripples which overlap the former are shown in Table 6. The relations between α and T for each seismogram in Table 6 are shown in Fig. 7. From this figure we can recognize the relation $\alpha = B(T) \cdot G(T) \propto T^{-n}$, $n = \frac{1}{2} \sim 1$. These results will confirm that Kanai's formula for B(T) is also appropriate in destructive earthquakes. It is interesting especially that the result from the seismogram of the Nobi earthquake at Nagoya Meteorological observatory shows the relation $\alpha \propto T^{-\frac{1}{2}}$, which is consistent with the envelope line of the maximum value of $\alpha(T) = B(T) \cdot G(T)_{T_0}$ for various T_0 in Fig. 1. This may be explained, if we consider that there are many predominant periods in the ground in Nagoya as the City is located on a very thick soft deposit which is likely to be composed of alluvium, diluvium and tertiary formations deposited in succession on hard paleozoic bed.

The spectrum of the earthquake motion occurred at upper part of Ibi River on Nov. 20, 1959 shows an exceptional trend in Fig. 11. The reason is probably found among the facts that the magnitude of the earthquake was small, the observing point was located on the mountain rock and Δ is small.

The seismograms of the Nobi earthquake obtained at Gifu and Nagoya are very precious from two reasons that firstly they were obtained near the origin of an earthquake of the greatest class (M = 8.4), and secondly that because of fast recording speed, we can very clearly read the vibrations with short periods and small amplitudes.

9. Conclusions

In order to determine the acceleration for the earthquake resistant structure design, we must investigate primarily, the expectancy of the maximum seismic intensity at the place during the life time of the structure and secondarily, the vibrational characteristics of the earthquake motion.

As to the primary problem, Kawasumi investigated the maximum seismic intensity to be expected at least once in a certain length of time at places throughout Japan. But the ground in the whole area is assumed to be the same as that of a standard ground. Mathematically, the maximum acceleration is expressed as

$$\alpha_{M,S} = B_M(T_{O,S}) \cdot G(T_{O,S})_{T_{o,s}} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (7)$$

In regard to the second problem, Kanai proposed formulas which are cited below with a slight modification of one in (3).

$$\alpha(T) = B(T) \cdot G(T)_{T_o} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (1)$$

$$B(T) = C(M, \Delta) \cdot T^{-1}, \quad \text{for } T \leqq T_m \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2)$$

$$\log T_m = 0.39M - 1.70 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (2')$$

$$G(T)_{T_o} = 4(1+\beta)^{-1} \cdot u^2\left\{(1-u^2)^2 + (\tau u)^2\right\}^{-\frac{1}{2}} \quad \cdots\cdots\cdots\cdots \quad (3)$$

$$\beta \doteqdot 1/5 \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (3')$$

$$\tau = 0.2/\sqrt{T_o} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (3'')$$

Therefore, by means of these results, and one additional knowledge of the predominant period of the ground, we can calculate the acceleration spectrum required for the design of the earthquake-proof construction by the following formula

$$\alpha_M(T)_{T_o} = B_M(T) \cdot G(T)_{T_o} = \frac{B_M(T) \cdot G(T)_{T_o}}{B_M(T_{O,S}) \cdot G(T_{O,S})_{T_{o,s}}} \times \alpha_{M,S} \quad \cdots\cdots \quad (8)$$

We tried in this paper to deduce an acceleration spectrum at Yokoyama Dam Site especially and examined the applicability of above formulas for destructive earthquakes from some points of view.

The results obtained in this paper are as follows:

(1) When we compare the seismic intensities at Yokoyama to those at Gifu in recent five destructive earthquakes and ten weak ones separately, the seismic intensities at Yokoyama for the destructive earthquakes were smaller (about 1/4 times) than those at Gifu, while for the weak earthquakes the former were larger (about 1.7 times) than the latter.

(2) We carried out a temporary observation by means of high sensitive seismographs and a routine observation by an ordinary seismograph at Yokoyama Dam Site. From the seismograms obtained we could see the predominant period at Yokoyama Dam Site is about 0.15 second. And from the seismograms obtained at Gifu Meteorological Observatory we could estimate the predominant period of the ground as about 0.5 second.

(3) Putting these values of predominant periods into Kanai's formula $G(T)_{T_0}$, and with due considerations of the periods to be particularly sensitive to human senses and to cause resonances of Japanese wooden houses, the phenomena shown in (1) are explained not only qualitatively, but also quantitatively. It is therefore to be noted that in case of destructive earthquakes, their accelerations on the ground in the mountainous area were not smaller than those on the ground in the alluvial plain, but the vibrations having short periods and large accelerations were left unnoticed in the mountainland owing to the lack of appropriate meters to estimate the seismic intensity of high frequency vibrations.

(4) Relying upon the result investigated by Kawasumi, the maximum acceleration to be expected at Yokoyama Dam Site at least once in 100 years is about 200 gals on a standard ground.

(5) Consequently, taking the vibrational characteristics into consideration and its period of the earthquake motion to be expected at Yokoyama Dam Site are calculated as about 300 gals and about 0.15 second.

(6) The acceleration obtained above seems to be rather large as compared to the values usually adopted in practice. It is primarily due to the smallness of the predominant period and the nature of earthquake motion that $B(T) \propto T^{-1}$. Although the earthquakes used by Kanai in the derivation of above relations had been restricted to rather weak ones, we were able to examine and verify the relations also in destructive earthquakes. Particularly from the seismograms of the Nobi earthquake of 1891 recorded at Nagoya and Gifu Meteorological Observatories we could make sure the relations above mentioned.

10. Acknowledgement

We express our thanks to Prof. H. Kawasumi in Earthquake Research Institute, Tokyo University for his constant guidance as well as for his valuable suggestions given to us in preparing the manuscript. Our thanks are also due to Dr. K. Kanai of the same institute for his kind guidance. We are very much obliged to the personnel of the research office of Yokoyama Dam, Chubu Chiho Kensetsu Kyoku for their help and cooperation offered to us throughout the course of present study. It is also to be remarked with thanks that Mr. S. Yokoyama, the principal of Ogaki Technical High School lent us the ordinary seismograph , and that Mr. Nakaoka kindly offered us the invaluable seismic data at Gifu Meteorological Observatory at our disposal. Lastly, but not the least, we must give our thanks to Mr. S. Kajita in our laboratory for his constant cooperation given to us in this study.

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Fig. 1 Acceleration spectra of an earthquake motion on various kinds of ground, $\alpha = B(T) \cdot G(T)$

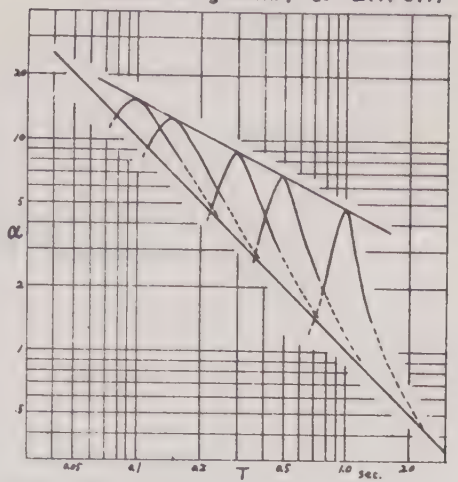

Fig. 2 Topographical map of central Japan and the location of Gifu and Yokoyama

Fig. 3 Subterranian Structure of Yokoyama dam site and locations of observation points

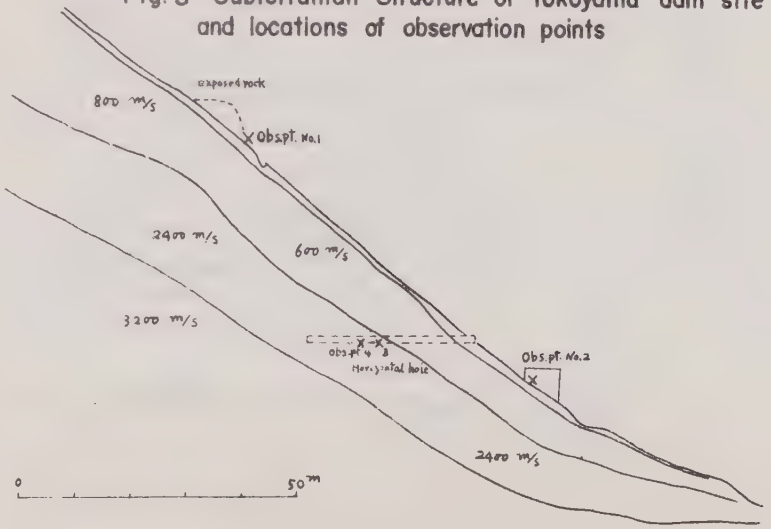

Fig. 4a Seismograms obtained at Yokoyama dam site

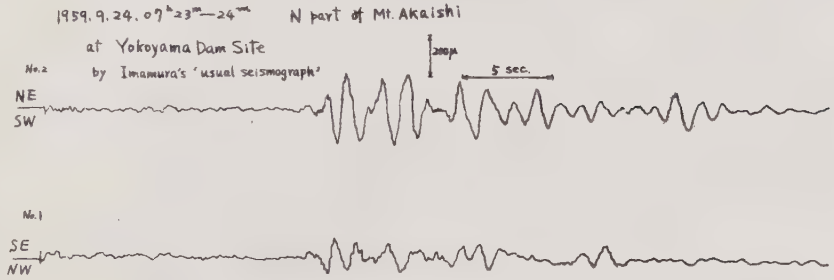

Fig. 4b

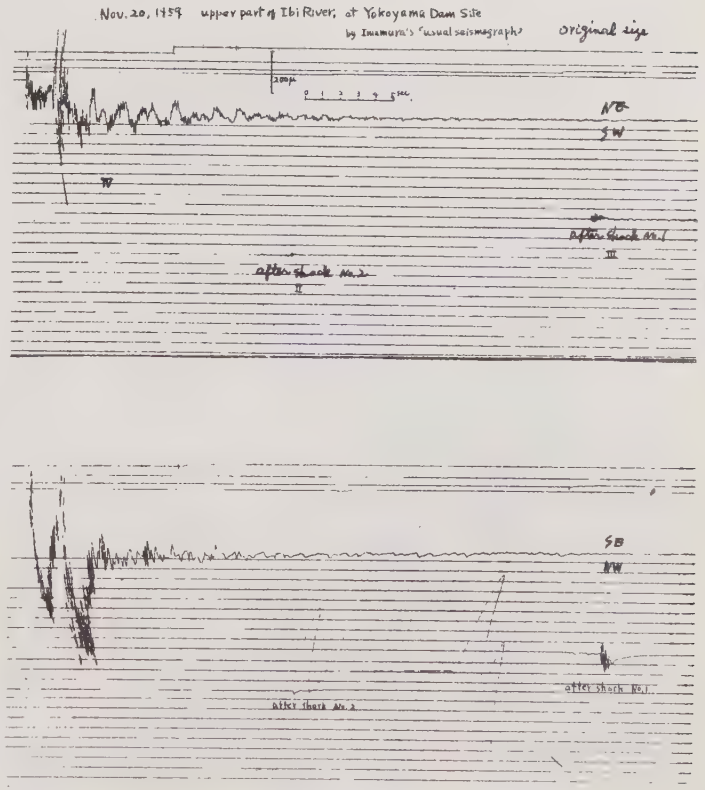

Fig. 4c Seismograms obtained at Gifu M. O. by Wiechert's seismograph

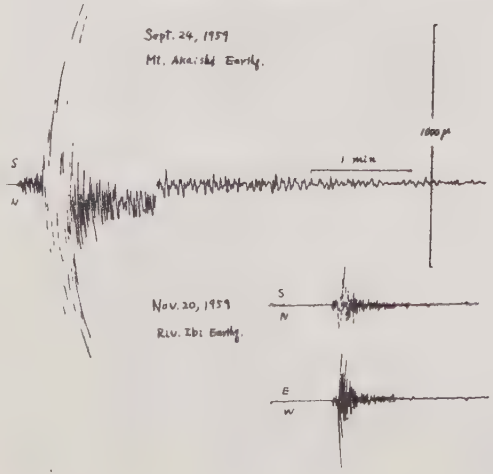

Fig. 5 Distribution of expectable maximum acceleration of earthquakes in 100 years in Japan (After Kawasumi).

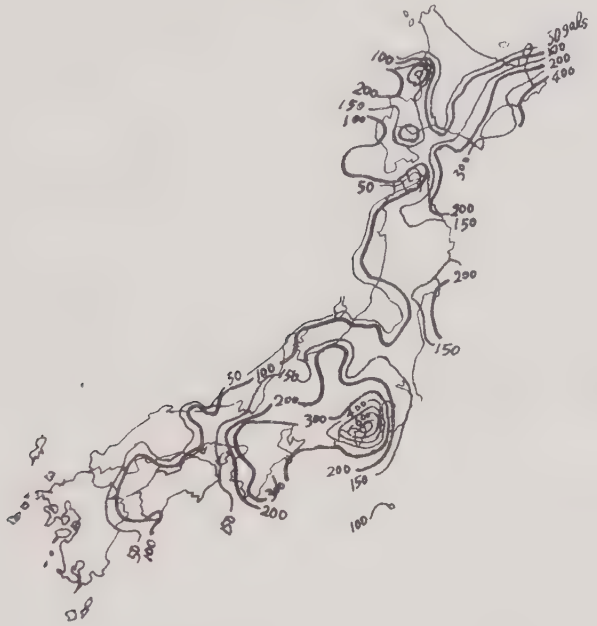

Fig. 6a Seismogram of The Nobi Earthquake at Nagoya M. O. Oct. 28, 1891

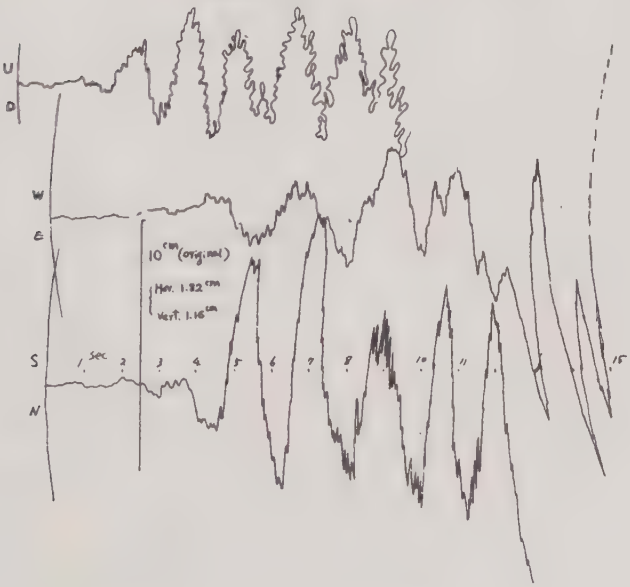

Fig. 6b Seismogram of The Nobi Earthquake at Gifu M.O. Oct. 28, 1891

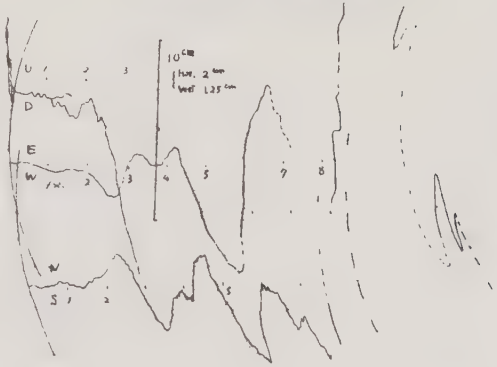

Fig. 6c Seismograms of The After Shock of The Nobi Earthq. Dec. 3, 1892

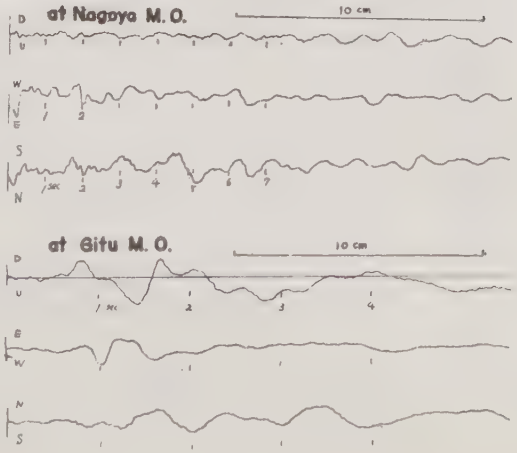

Fig. 7 Acceleration spectra obtained from seismograms of great earthquakes

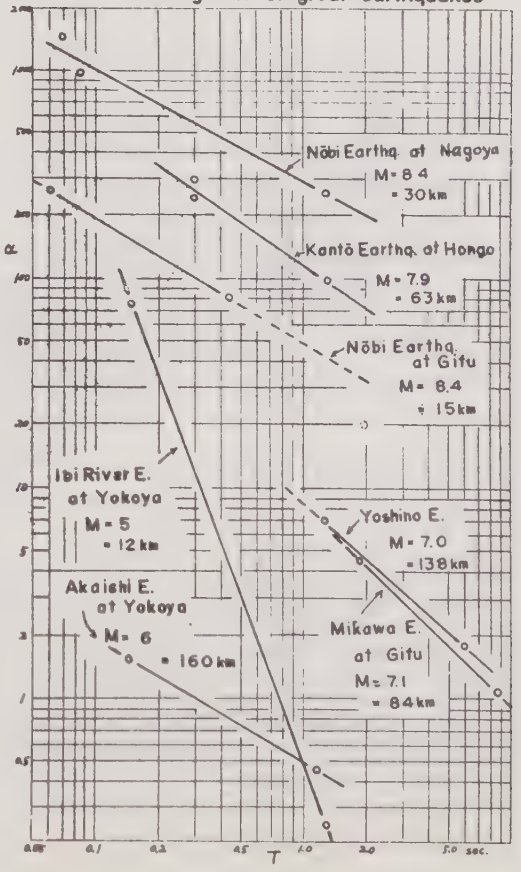

Table 1 Comparison of the seismic intensity at Yokoyama with that at Gifu

Earthquake	Time	φ	λ	D	M	I G	I Y	Δ G	Δ Y	A G	T G	$(\alpha\Delta^2)_Y/(\alpha\Delta^2)_G$	$(\alpha\Delta)_Y/(\alpha\Delta)_G$
				km				km	km	mm	sec.		
Yoshino Earthq.	1952. 7. 18	34.45	135.8	70	7.0	4	3	138	138	18.5	1.85	0.367	0.340
Fukui Earthq.	48. 6. 28	36.1	136.2	20	7.3	4	3	100	68	41.3	1.87	0.146	0.210
Nankai Earthq.	46.12.21	33.0	135.6	30	8.2	5	3	284	298	out	3.4	0.110	0.105
Mikawa Earthq.	45.1.13	34.7	137.0	0	7.1	4	3	84	114	28.7	1.55	0.590	0.425
Tonankai Earthq.	44.12. 7	33.7	136.2	0~30	8.0	5	4	196	210	out	2.5	0.304	0.340
												0.263	0.260
E. Aichi	1958. 5. 4	35.0	137.4	50	4.8	2	2	73	105	190"	0.4	2.06	1.445
M. Gifu	58. 2. 9	35.7	137.2	0	4.3	2	1	51	68	84	0.3	0.56	0.435
M. Fukui	57.12.31	35.8	136.1	20	4.6	2	2	75	43	133	0.35	0.335	0.564
Gifu-shiga border	57. 12. 3					1	3	36	13.5	72	0.44	1.27	5.67
W. Mie	57.11.27	34.7	136.2	70	4.6	1	2	93	99	89	0.36	3.68	3.47
W. Aichi	56.12.18	35.2	136.7	40	4.4	1	2	23	46	98	0.44	13.2	6.55
S. Aichi	56.12.11	34.9	137.1	30	4.0	1	1	63	94	25	0.3	2.25	1.46
M. Aichi	56. 2. 8	34.9	137.2	20	5.0	3	3	88	100			2.21	1.43
W. Gifu	52. 6. 26	35.5	136.6	30	4.7	2	3	19	14			1.58	2.3
S. Mie	51. 9. 23	34.3	136.2	40	4.8	1	1	134	143			1.18	1.09
												1.72	1.73

Table 2 Seismic observation at Yokoyama Dam Site

Routine observation of earthqakes

Observation point	Instrument	Results
On the surface layer (on the concrete base in the observation hut)	Imamura's usual seismograph Horizontal 2 component $T_0=2^{sec}$, $h\fallingdotseq0.5$, $V=63$, p.s.=$5.32\frac{mm}{sec}$ With starter and stopper Smoked paper recording	Since Aug. 1958 to the present time untelt earthqaake No.325 felt. III, Aug. 22, 1959 felt. I~II, Aug. 31, " felt. I, Sept. 24, " felt. IV, Nov. 20, "
On an exposed rock	Electro magnetic seismograph with amp. MG II, Trancducer N.D. Hor. NS $T_0=0.80^{sec}$, $h\fallingdotseq0.5$, $V=0.022\frac{mm}{\mu kine}$ p.s.=$8\frac{mm}{sec}$ Smoked paper recording	From evening to next morning Aug. 29~Dec. 10, 1958 335 micro earthquakes were recorded

Temporary observation (1) Observation of the ground motion due to the explosion of 150 ton dynamite at Miboro

Observation point	Instrument
	Electromagnetic seismograph with amp. MG II, p.s. $34\frac{MM}{SEC}$
On the surface layer	$T_0=0.455^{sec}$, $h\fallingdotseq0.5$, $V=0.12\ ^{mm}/_{\mu kine}$, Hor. NS
On the exposed rock	$T_0=0.80$ ", $h\fallingdotseq0.5$, $V=0.50$ ", Hor. NS
In the horizontal hole (compact rock, velocity of P wave=2400 m/sec)	$T_0=0.475$ ", $h\fallingdotseq0.5$, $V=0.37$ ", Hor. NS
In the horizontal hole (loose rock, V_p=2400 m/s)	$T_0=0.27$ ", $h\fallingdotseq0.5$, $V=0.06$ ", Hor. NS

(2) Observation of the ground noise

Observation point	Instrument
In the horizontal hole (compact rock)	Electromagnetic seismograph of which electric power is supplied by batteries $T=0.80^{sec}$, $h\fallingdotseq0.5$, $V=4.\ ^{mm}/_{\mu kine}$, p.s. = $50\ ^{mm}/_{sec}$

Table 3 Predominant periods obtained by micro-seismic observation at Yokoyama Dam Site

Predominant periods obtained from the ground motion caused by the great explosion of 150 tons dynamite at Miboro Dam. $\Delta = 70^{KM}$

Observation point	Initial part	Main part	Tail
On the surface layer	0.12 sec.		(0.036) 0.13
On the exposed rock	0.12	0.14	0.175
In the horizontal hole (compact rock)	0.175		(0.070) 0.215
In the horizontal hole (loose rock)	0.175		0.175

Predominant period obtained from 335 micro-earthquake picked up on the exposed rock is $T_0 =$ 0.15 sec.

Pred. periods obtained from ground noise on the surface layer, $T_0 =$ (0.036) and 0.15 sec.

Pred. period obtained from ground noise in the horizontal hole (compact rock) $T =$ 0.10 sec

Table 4 Predominant periods obtained from the seismograms of weak earthquakes obtained by 'usual seismograph' on the surface layer

Time	Δ	Short period NE/SW comp.	Short period SE/NW	Long period NE/SW	Long period SE/NW
Aug. 22, 1959	about 12 km	0.10 sec	0.11	1.0	3.75
Aug. 31, 1959	12	0.103	0.115	1.6	3.8
Sept. 24, 1959	160	0.15	0.15	1.2 1.6	1.12
Nov. 20, 1959	11	(0.10) 0.15	(0.10) 0.23	1.37	1.8

Table 5 Predominant periods obtained from the seismograms of destructive earthquakes, weak earthquakes and ground noise at Gifu Meteorological Observatory. By Wiechert's seismog.

Earthquake		Predominant period		
Fukui Earthquake (by Wiechert Seismograph)		0.47 sec.	1.29	6.2
Mikawa Earthquake			1.1	9.3
Yoshino Earthq.			1.3	6.5
Nobi Earthq.	0.06	0.445	2.0	
Weak Earthquakes		0.3—0.44		
Ground Noise	0.09, 0.2	0.5		
Ground Noise (by electro-magnetic seismog.)	0.05, 0.07			

Table 6 Acceleration spectra obtained from seismograms of great earthquakes

Earthq.	Epic.	Time	M	T_M	Obs.pt	Δ	I	T_{short}	As	α_S	T_{long}	A_L	α_L	Seismograph T_0	V	v	p.s.
Nobi E.	WNW of Gifu	Oct. 28 1891	8.4	sec 4.0	Gifu	km 15 ~	6 ~7	sec 0.06 0.444	cm 0.024 0.410	gal 270 84	sec 2.0	cm >2	gal >20	sec 2 ~3	Miln's s. 5 (8)		no 21.4
Nobi E.	WNW of Gifu	Oct. 28 1891	8.4	4.0	Nagoya	30 ~	6 ~7	0.067 0.084	0.175	1560 990	1.3	(11)	260	2 ~3	Miln's s. 5.5 (9)		no 14.6
Kanto E.	35.33 139.33 ?	Sept. 1 1923	7.9	25	Hongo	63	5 ~6	0.30	0.6 0.8	250 300	1.35	4.43	100	?			
Mikawa E.	34.7 137.0 0	Jan. 13 1945	7.1	12	Gifu	64	4	could not read for the slow paper running speed			1.1 9.3	0.45 2.4	14.9 1.11	Imamura's strong s. 6	2	no	0.6
Yoshino E.	34.45 135.8 7.0	July 18 1952	7.0	11	Gifu	138	4	could not read for slow p.s.			1.3 6.5	0.3 1.9	7.1 1.8	Imamura's strong s. 6	2	no	0.6
Mt. Akaishi E.	35.6 138.2 20	Sept. 24 1959	6	5	Yokoyama	160	1 ~2	sec 0.15	μ 8.75	gal 1.55	1.2	0.017	0.47	Imamura's usual s. 1.8	6.3	2.5	5.32
Riv. Ibi E.	35.6 136.6 20	Nov. 20 1959	6	2	Yokoyama	12	4	0.15	μ 435	77	1.37	0.012	0.26	Imamura's usual s. 1.8	6.3	2.5	5.32

GROUND MOTIONS GENERATED BY UNDERGROUND NUCLEAR EXPLOSIONS

By Dean S. Carder and William K. Cloud

The U. S. Coast and Geodetic Survey measured ground effects from a number of large underground nuclear explosions of the PLUMBBOB (Rainier) and HARDTACK II series and certain other tests using high explosives. In this article some of the results of engineering interest will be discussed.

The nuclear tests under discussion were fired in massive tuff beneath the Rainier mesa of the Nevada Test Site during September 1957 and October 1958. These tests are described in the literature (Johnson et al, 1959). Their estimated yields and other pertinent data are summarized in Table I.

In Figure 1, shot locations are indicated by stars accompanied by the initials of the respective shots. The station locations during the HARDTACK II series (October 1958) are indicated by numerals, the last two digits of the official station numbers. Station 1200.14 is indicated by .14, and station 1200.01 by .01, etcetera. Figure 2 is a cross section from Station .14 on the mesa to Station .01 on the quartzite, a few miles east of the mesa. Not shown are Stations .09 on granite and 7.2a[2] on deep alluvium. Geologic structure of the area is indicated. The station layout during shot Rainier was covered in an earlier report (Carder and Cloud, 1959). The station locations shown here and in the earlier report are compromises between scientific optimum and economic feasibility.

The instruments consisted of the standard strong-motion seismographs used for years by the Survey for measuring earthquake motion (Cloud and Carder 1956). They are direct recording seismographs consisting of simply constructed compound pendulums damped by permanent magnets together with requisite timing and photo-mechanical recording apparatus. These instruments are capable of recording ground motion in the acceleration range from 0.001 g or less to 3.0 g, and transitory ground displacements of 6 inches (15 cm) or less. For this purpose two sets of seismometers are required, one set having pendulum periods substantially less than the periods of expected ground motion and the other with periods substantially greater than the ground periods. Results of earlier work served as guides to optimum gain settings of the instruments.

The authors are of the U. S. Department of Commerce, Coast and Geodetic Survey, and are, respectively, Chief Seismologist, and Chief, Seismological Field Survey.

Test Results

Formulas which prior to the HARDTACK II series seemed to give satisfactory results are

(1) $\log a = 0.75 \log W + 4.0 - 2 \log D$

(2) $\log A = 0.75 \log W + 4.6 - 2 \log D$ (0.3 to 3 km)

(3) $\log A = 0.75 \log W - 2.9 - \log R - 0.006 R$ (3 to 160 km)

Where a = maximum acceleration, single component (gravity)

A = maximum displacement, single amplitude, single component (centimeters)

W = TNT equivalent of explosion in tons

D = distance to explosion (feet)

R = distance to explosion (kilometers)

Figures 3 and 4 show tracings of acceleration records from the Blanca and Logan shots, and Figures 5 and 6 show traces of corresponding displacement records. Figure 7 is a plot of maximum observed accelerations with distance. The solid graphs are according to predictions. Figure 9 is a corresponding plot of maximum observed displacements and the graphs are based on predictions. The fit of observed data to predicted values is in general fair considering that predictions are extrapolations from 10 to 50 tons of HE to 19,000 tons equivalent of nuclear energy. There are discrepancies, however. For example, the Tamalpais acceleration data are a factor of 5 or 6 higher than were the predicted values and Evans displacement data from stations beyond a few miles are at least an order too low.

In the evaluation of the evidence of Figure 9, one should be aware of the physical location of each station in respect to the shot, and the geologic underground near the station. To assist with this evaluation, strong-motion data are designated by the initial of the rock upon which the station is located, or by the initial accompanied by a symbol.

The data may be interpreted as follows:

1. Ground amplitudes on deep alluvium are higher than amplitudes at like distances on granite by factors of 2 to 3.5.

2. Amplitudes on granite, quartzite, and limestone are generally consistent

with each other and generally predictable with the use of the second leg of the displacement formula (equation 3), applicable in the formula for distances from 2 to 100 miles, but in this case from 0.5 to 100 miles.

3. Amplitudes on bedded tuff are not consistent, most of the time they are high but in some cases they are comparable with data on limestone, etc.

4. Amplitudes on welded tuff are always high. This is more likely a combination of a geographic condition and the nature of the rock. The welded tuff caps the mesa containing the shots, and the stations on the mesa (welded tuff) are higher in elevation than the shots. The stations on the top and side of the mesa doubtless receive seismic energy from the shots in a more concentrated form, whereas, at the other stations there may be some loss by refraction.

5. Amplitudes at teleseismic stations are scattered but follow predicted estimates within a factor of 3. From item (2), it appears that earlier postulates which imply that amplitude falls off as the square of the distance out to 3 km was an apparancy because of the geographic position of the stations; and the condition that some of the stations were located on top of the mesa where they were in a position to receive a greater concentration of energy. If the data pertaining to a given rock unit, limestone for example, are examined more closely, it appears that amplitudes attenuate linearly with distance - with absorption. This is in accordance with the idea that elastic energy is propagated from the source in spherical wave fronts, assuming no change in frequency. Energy in this form attenuates as the square of the distance.

We have the simple formulas:

$$E \propto v^2, \quad v = \frac{2\pi}{T} A; \quad a = \frac{2\pi}{T} v$$

where v is the particle velocity during a passing wave front and E, a, T, and A are as defined elsewhere. If absorption is negligible, it follows that A would be expected to attenuate linearly with distance assuming no change in period, and aT would likewise attenuate linearly with distance. According to observation, waves associated with the maximum displacements maintain about the same period from place to place, but maximum accelerations are associated with short periods near the source, and with longer periods at relatively larger distances. Maximum accelerations and their associated periods have been combined as a product and plotted according to distance in Figure 8. The estimate of these associated periods is rough, which alone may account for the scatter of the data. Nevertheless the trend toward linear attenuation is evident.

Energy contained in seismic waves in an assumed spherical wave front at a

distance of about 1 km from the Blanca shot was calculated using the initial crests recorded on the displacement records and applying the following formulas:

$$E_s \text{ (ergs/cm}^2\text{)} = 0.5\pi^2\rho \sum VA^2/T \quad \text{and}$$

$$E \text{ (ergs)} = 2\pi E_s R^2 \times 10^{kR}$$

Here E_s is the energy density in the wave front as it passes a station; E is the total energy in the wave front, assuming that it maintains a spherical shape from the source to the station; ρ is the rock density; V is the seismic wave speed in the rock; A is the zero to peak ground amplitude; T is the wave period; and k is an absorption factor, base 10. An additional catch-all factor which includes loss by refraction is normally applied, but since in this case it is considered unity, it was not included.

Using data from the 10 nearest C&GS stations, the calculated log E of the Blanca shot is 18.46± 0.15 which is equivalent to a magnitude 4.8 earthquake. Since the calculated yield of this shot was 19 kt, or the equivalent of $10^{20.9}$ ergs, the estimated energy in the wave front 1 km from the source is about 0.4 percent of the total yield. Much seismic energy may have been absorbed near the source, but the value given here is pertinent when comparisons with earthquakes are made.

Trajectories of particle motion during the passage of seismic waves from shot Blanca across two selected stations are shown in Figures 10 and 11. The trajectories are represented in three planes (a) the vertical plane containing the ray, (b) the horizontal plane, and (c) the vertical plane at right angles to the ray. In some cases, the initial part of the motion is shown on an enlarged scale. The selected stations are .05 on top of the mesa at a distance of approximately 5,500 feet, and station .08 on limestone at a distance of 9,460 feet.

It is believed that the initial wave at each station had part of its history in the limestone underlying the tuff in which the shots were fired. This wave is identified as P_L. It must be, and is, essentially vertical and radial. An impulse following P_L by a few tenths of a second likewise has a strong radial direction. For stations on the mesa, the direction is in phase with the initial P_L and is believed to be associated with the direct, or almost direct, wave from the source to the station through the tuff. This wave is designated P_T. It arrives at Station .05 about 0.85 sec after zero time, compared with 0.55 sec for P_L. At Station .08 the second impulse is oppositely directed from the first, that is, it is out of phase with the first arrival. It arrives about 0.25 sec later, and is believed to be a reflection of the P wave on the surface. The longer part of its path in the slow speed tuff is believed to be responsible for the delay in arrival time of 0.25 sec. It is designated pP meaning a P wave from the source to the surface near the source and then reflected as a P wave to the station. This wave has comparatively high energy

and is believed responsible in part for the confusion in identifying some first arrivals as positive upward at distant points. The corresponding S_L, ST, and sS arrivals are indicated on the illustrations.

RESPONSE SPECTRUM ANALYSIS OF RECORDS FROM EVENTS BLANCA AND LOGAN

By William K. Cloud

A response spectrum is defined as the envelope of maximum relative velocities obtained by a series of oscillators when subjected to known accelerations. As pointed out by Hudson in 1956 (Reference 1), such spectrums are useful not only for the determination of structural responses in particular cases of ground motion, but also for investigating the general character of ground motion.

Within the past 20 years, records from a number of earthquakes, and several quarry blasts, have been subjected to response spectrum analysis (References 2, 3, 4, 5, 6). Event Blanca (19 kilotons) and Logan (5 kilotons) offered an excellent opportunity to make similar analyses of records from underground nuclear explosions.

Relative maximum velocity spectrums from 11 radial component acceleration records of these two events were run on a Mark II Electric Analog Spectrum Analyzer, described by Caughey, Hudson, and Powell elsewhere in the Proceedings of this Conference.

Ten of the resultant spectrums are shown in Figures 12 to 21. For interpretation of earthquake spectrums Benioff (Reference 7) and Housner (Reference 8) suggest use of "Response Spectrum Intensity" defined as the area under the relative velocity response curve between oscillator periods of 0.1 and 2.5 sec and thus a measure of an average spectrum value over the range of periods of structural interest. With slight modification, the suggested method has been used for interpretation of the nuclear detonation spectrums. Results are given in terms of the average maximum velocity for the oscillator period range 0.1 to 2.0 sec rather than in terms of area.

At first glance spectrums in Figures 12 to 21 show considerable variation. However, when average maximum velocities for 10 percent damped oscillators are plotted against station distance from detonation (Figure 22), attenuation appears reasonably uniform. Data from Station 1200.07 are exceptions for which there is no immediate explanation.

Disregarding the exceptions, Figure 22 suggests the possibility of developing an empirical formula relating seismic energy to average maximum spectrum

velocity and to distance. Assume a formula of the following type:

$E = KVD^n$ Where D = slant distance to nuclear detonation (feet)

V = average maximum spectrum velocity at distance "D" (feet/sec). (Period range 0.1 to 2.0 sec, 10 per cent critical damping.)

E = total seismic energy (ergs)

K and n = constants

The slope of the lines in Figure 22 indicates that n = 1.78. Corresponding values of V and D read from the Blanca line in Figure 22, combined with Carder's value for seismic energy of Blanca ($10^{18.4}$ ergs), gives a value for K of $10^{12}/2.4$. Substitution gives

$$E = \frac{10^{12} VD^{1.78}}{2.4}$$

A comparison of seismic energies calculated by the formula and by other methods for Logan and four earthquakes is shown in Table 2. Many additional strong-motion records from underground detonations or earthquakes are needed to judge whether or not the formula has merit. The order difference between energy calculated by the formula and given by seismologists for two of the earthquakes is certainly not encouraging. However, in both cases the formula value does back up observation. Engineers noted that, considering magnitude ratings, effects on structures were below average for the Hebgen Lake shock; above average for the Port Hueneme shock.

For comparison with spectrums from the nuclear detonations, two earthquake spectrums are shown in Figure 23. A verbal communication from Dr. Don Tocher, University of California seismologist, placed the epicenter of the earthquake approximately 4.9 miles from the Hollister station and gave the magnitude as 5.0. Average maximum velocities (10 per cent critical damping) for the oscillator period range 0.1 to 2.0 sec were: N 81° W component, 0.290 ft/sec; N 01° E component, 0.203 ft/sec. Modified Mercalli scale intensity at Hollister was VI.

Tentative conclusions may be drawn from comparing the data:

a) Although there is some dissimilarity between the earthquake and detonation spectrums this dissimilarity is no greater than between the two earthquake spectrums from the same stations or between spectrums of the same detonation at different stations.

b) Buildings at distances somewhere between 2.4 and 3.0 miles from the Blanca detonation and 1.6 and 1.9 miles from the Logan detonation would have been subjected to similar ground effects as the building at Hollister located 4.9 miles from the epicenter of the 19 January 1960 earthquake. In this connection it is of interest to quote from a report by observers stationed 2.75 miles from the Blanca detonation:

"Immediately following zero time as indicated by the countdown a sharp light reflection, somewhat resembling a light flash in appearance, was observed for several hundred yards along the profile of the mesa as the initial shock wave passed through the earth's surface. This was followed by a severe earth tremor, resulting in an up-and-down motion similar in appearance to water wave action. Vehicles were observed to rock or bounce on their springs. This severe tremor lasted approximately 3 seconds, after which it subsided to a gentle rolling movement which persisted approximately 10 seconds."

ACKNOWLEDGMENTS

The research projects, on which this paper is based, were carried out by the Coast and Geodetic Survey for the U. S. Atomic Energy Commission under Memorandum of Understanding AT (29-2)-746 dated September 2, 1958, and Supplemental Agreements.

Acknowledgments are due others of the staff of the Coast and Geodetic Survey for collection and assembly of field data. Special mention is due E. B. Roberts, Research Advisor, and R. A. Earle, Chief, Division of Geophysics; and to L. M. Murphy, W. V. Mickey, T. H. Pearce, John Hershberger, B. J. Morrill, R. P. Maley, C. F. Knudson, and A. E. Lamison.

Further acknowledgments are due to the following persons outside the Coast and Geodetic Survey who contributed much to the success of the projects covered by this paper: R. G. Preston, Don Schueler, Tom Brockett, and S. E. Warner, University of California, Lawrence Radiation Laboratory; G. W. Housner, and D. E. Hudson, California Institute of Technology, and Perry Byerly, University of California.

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12. Cloud, W. K. and Carder, D. S., "The Strong-Motion Program of the Coast and Geodetic Survey," Proceedings of the World Conference on Earthquake Engineering, Berkeley, California, June 1956.

Figure 1. Location of strong-motion seismograph stations.

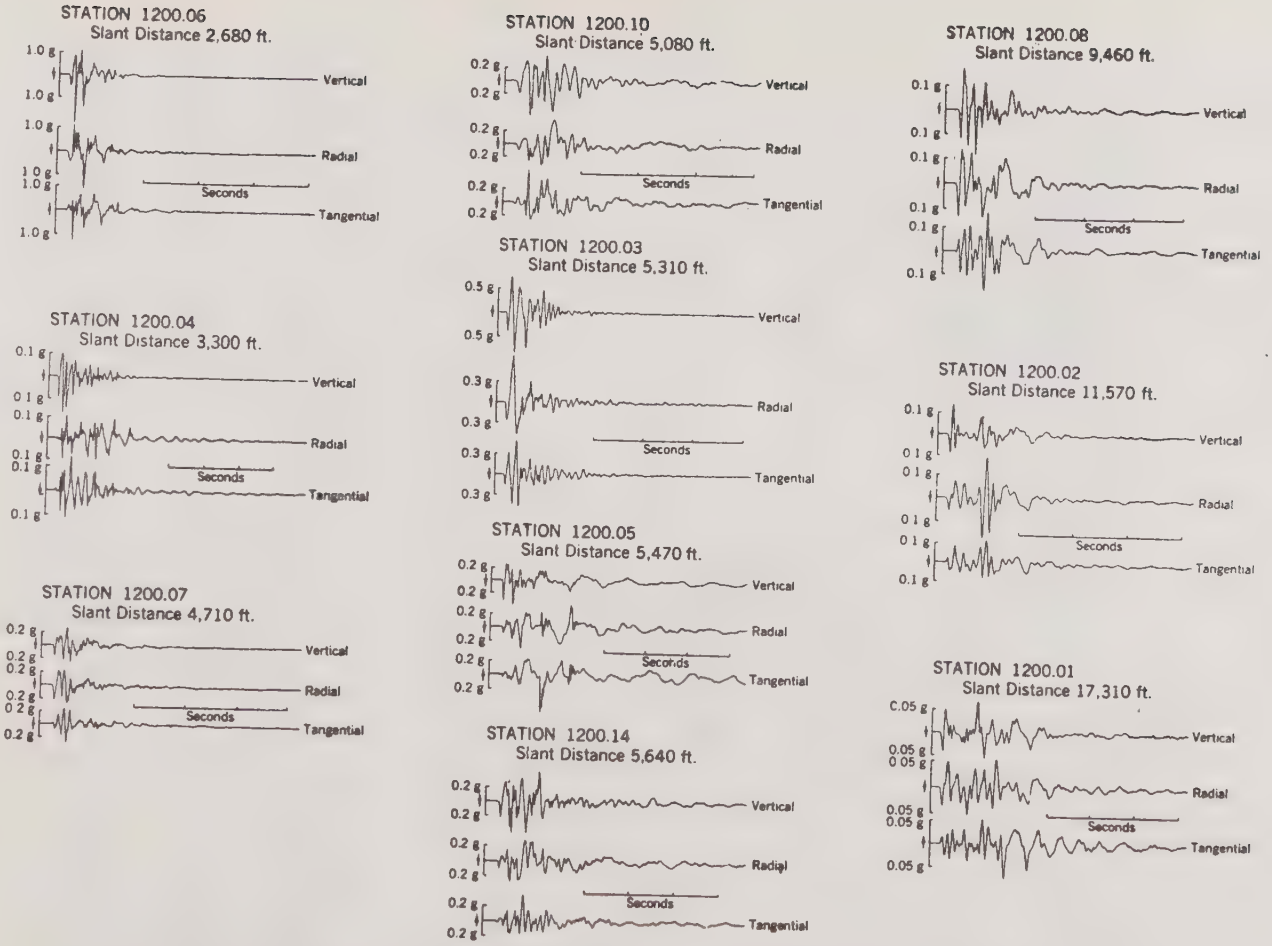


Figure 3--Tracings of acceleration records, Blanca Shot. The direction of the arrows indicate initial ground motion up, radially away from zero, and tangentially to the left when facing zero.

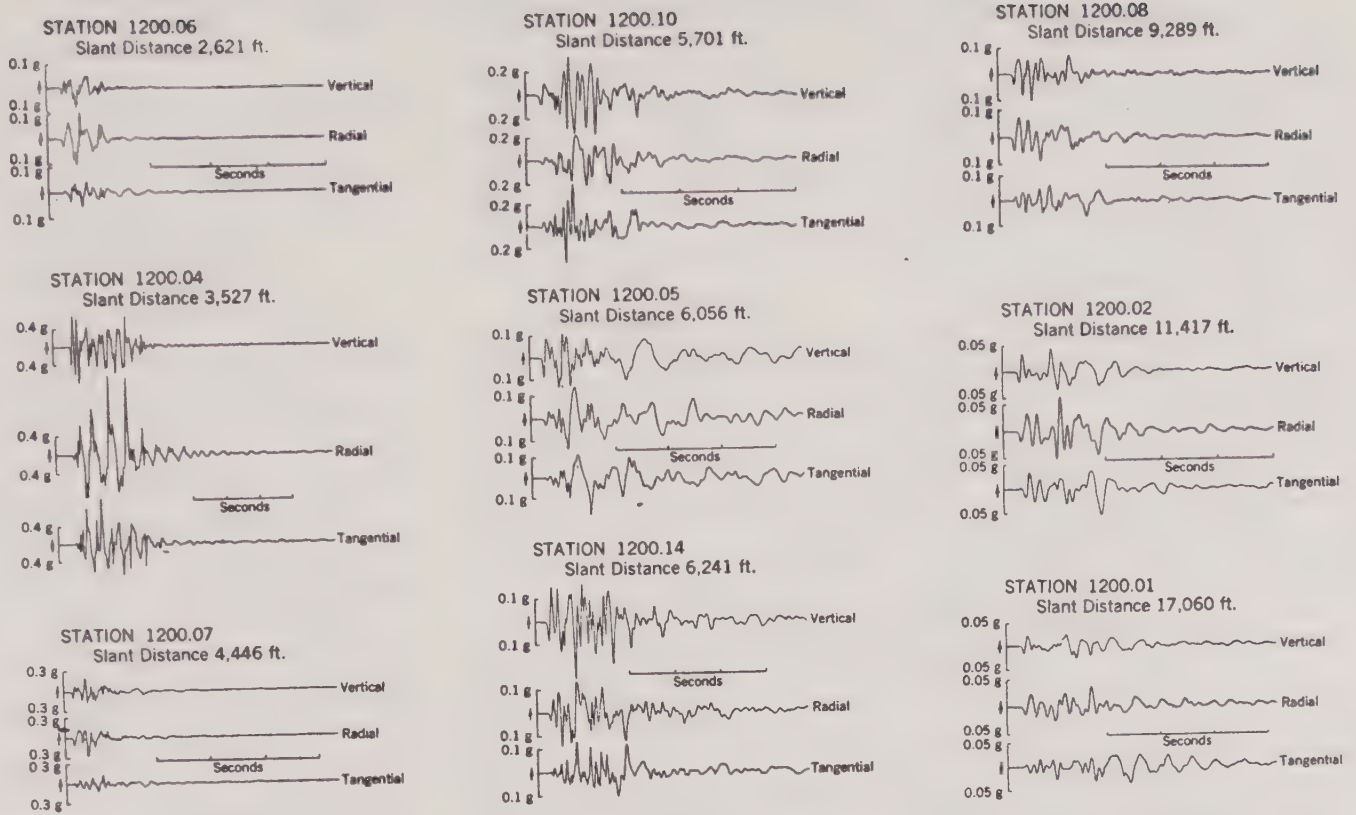


Figure 4--Tracings of acceleration records, Logan Shot. The direction of the arrows indicate initial ground motion up, radially away from zero, and tangentially to the left when facing zero.

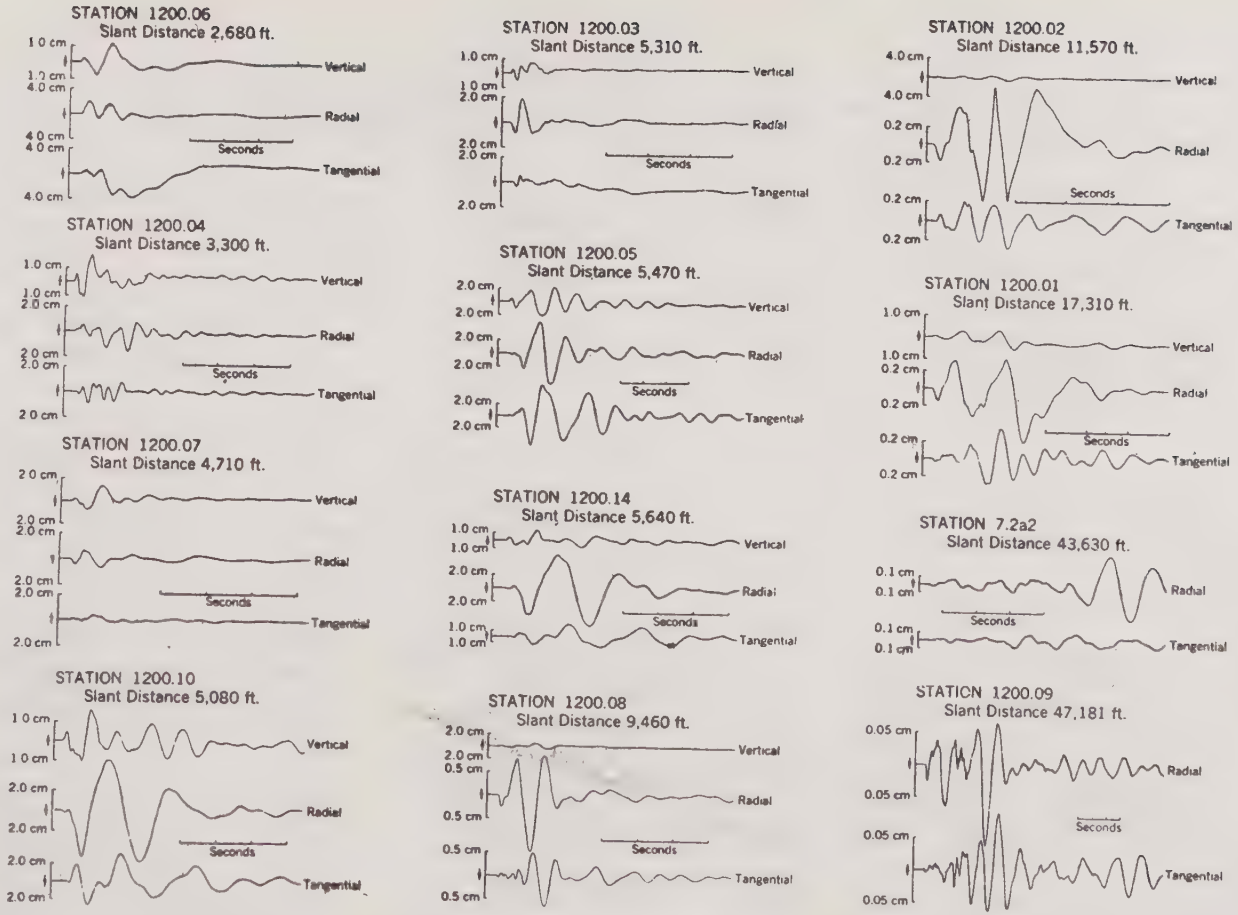


Figure 5--Tracings of displacement records, Blanca Shot. The direction of the arrows indicate initial ground motion up, radially away from zero, and tangentially to the left when facing zero.

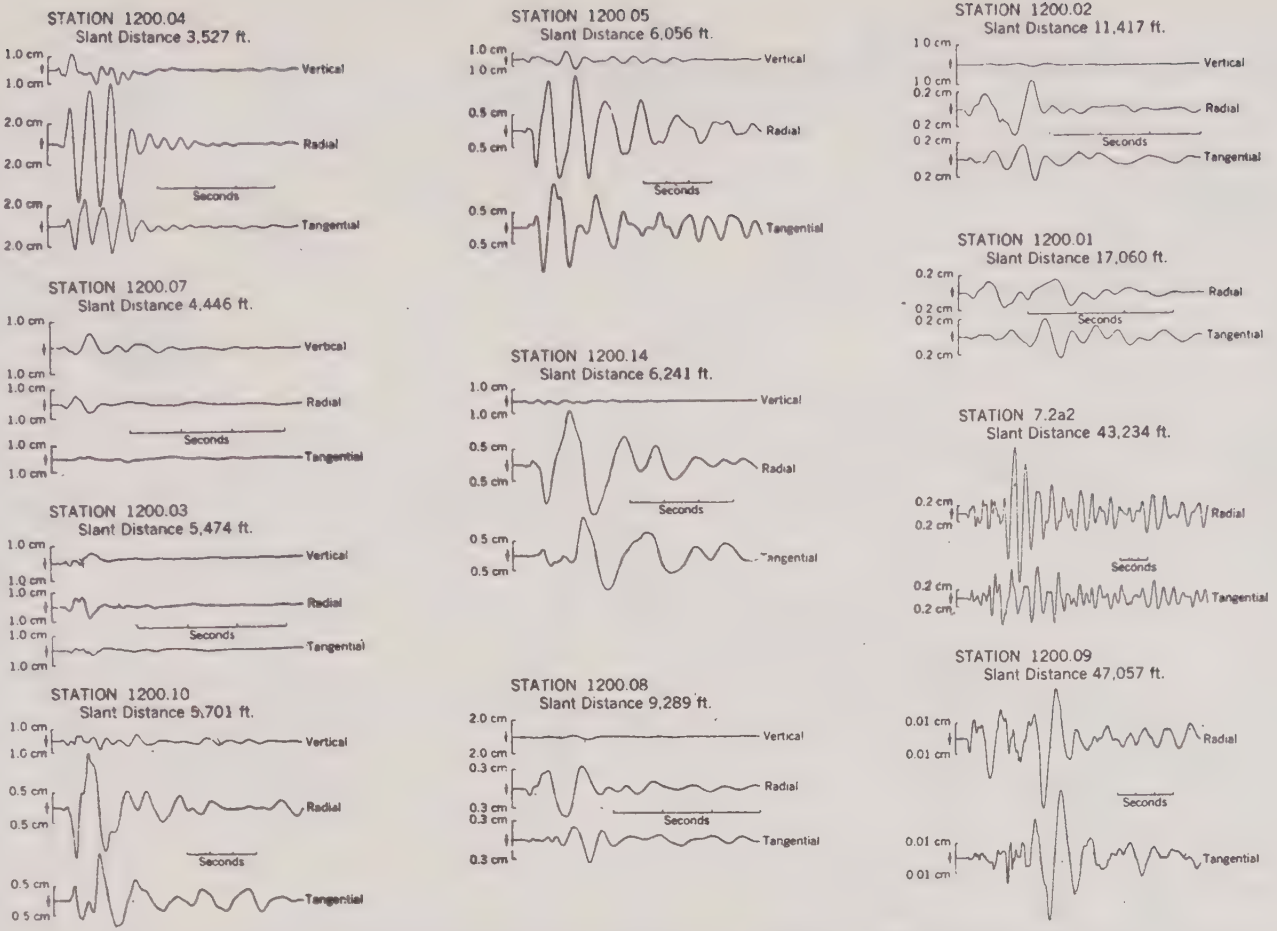


Figure 6--Tracings of displacement records, Logan Shot. The direction of the arrows indicate initial ground motion up, radially away from zero, and tangentially to the left when facing zero.

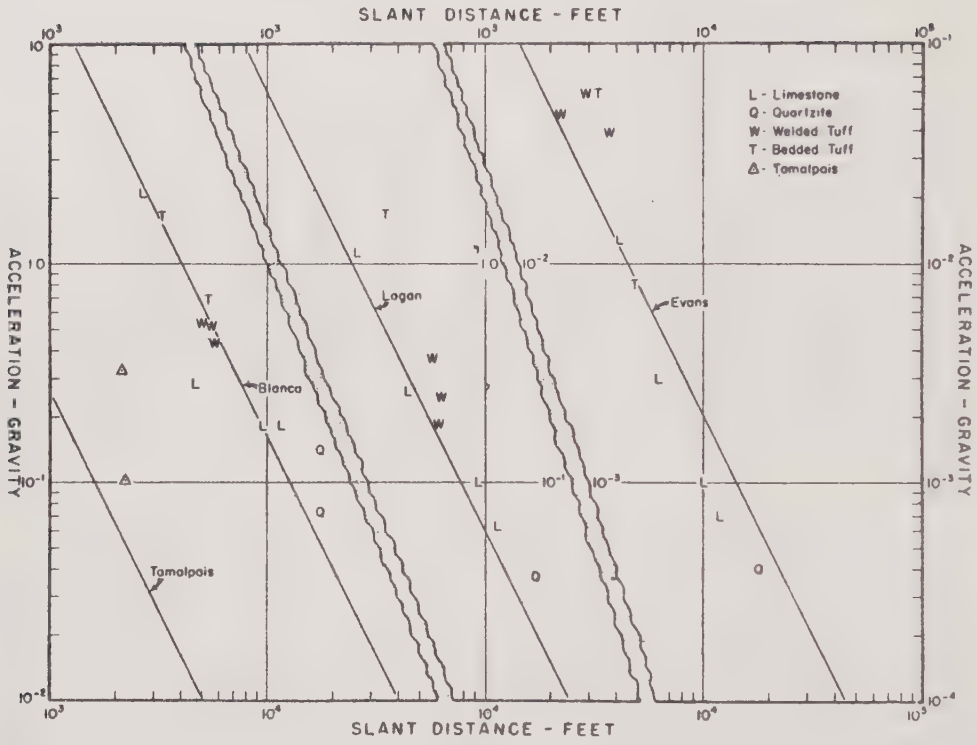


Figure 7--Attenuation of ground acceleration with distance.

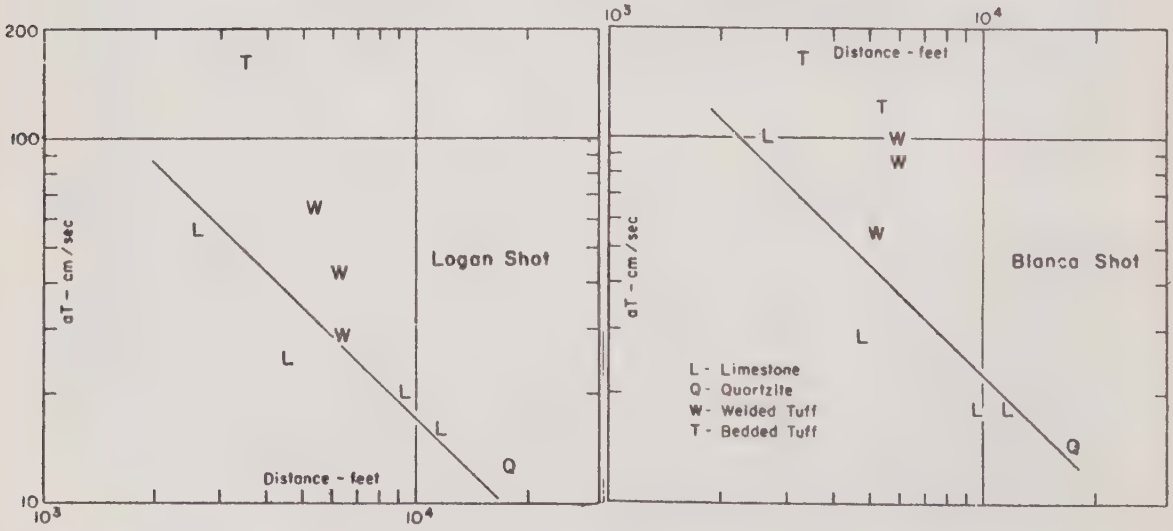


Figure 8--Attenuation of the product of acceleration and corresponding period with distance.

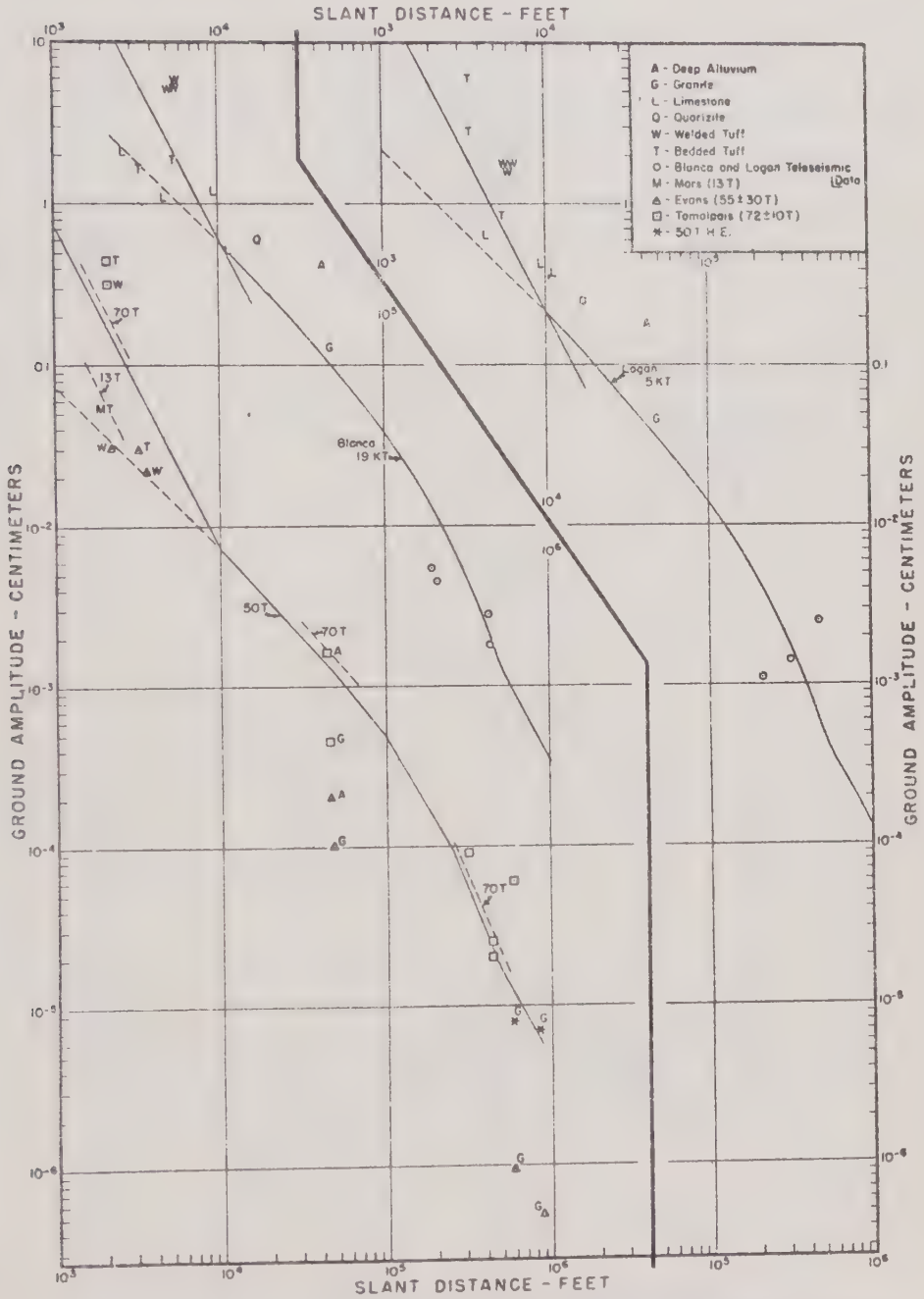


Figure 9--Attenuation of ground displacement with distance.

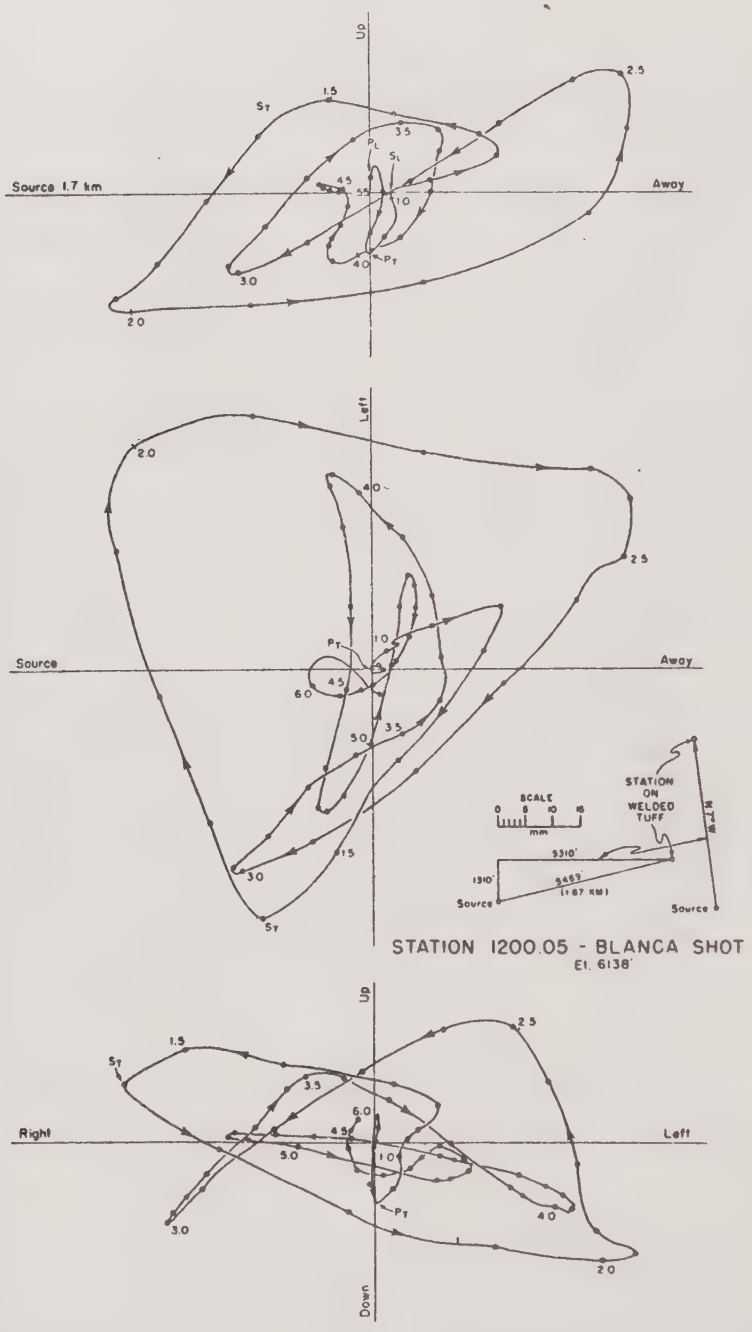


Figure 10--Trajectory of particle motion - Blanca Shot. Station 1200,05

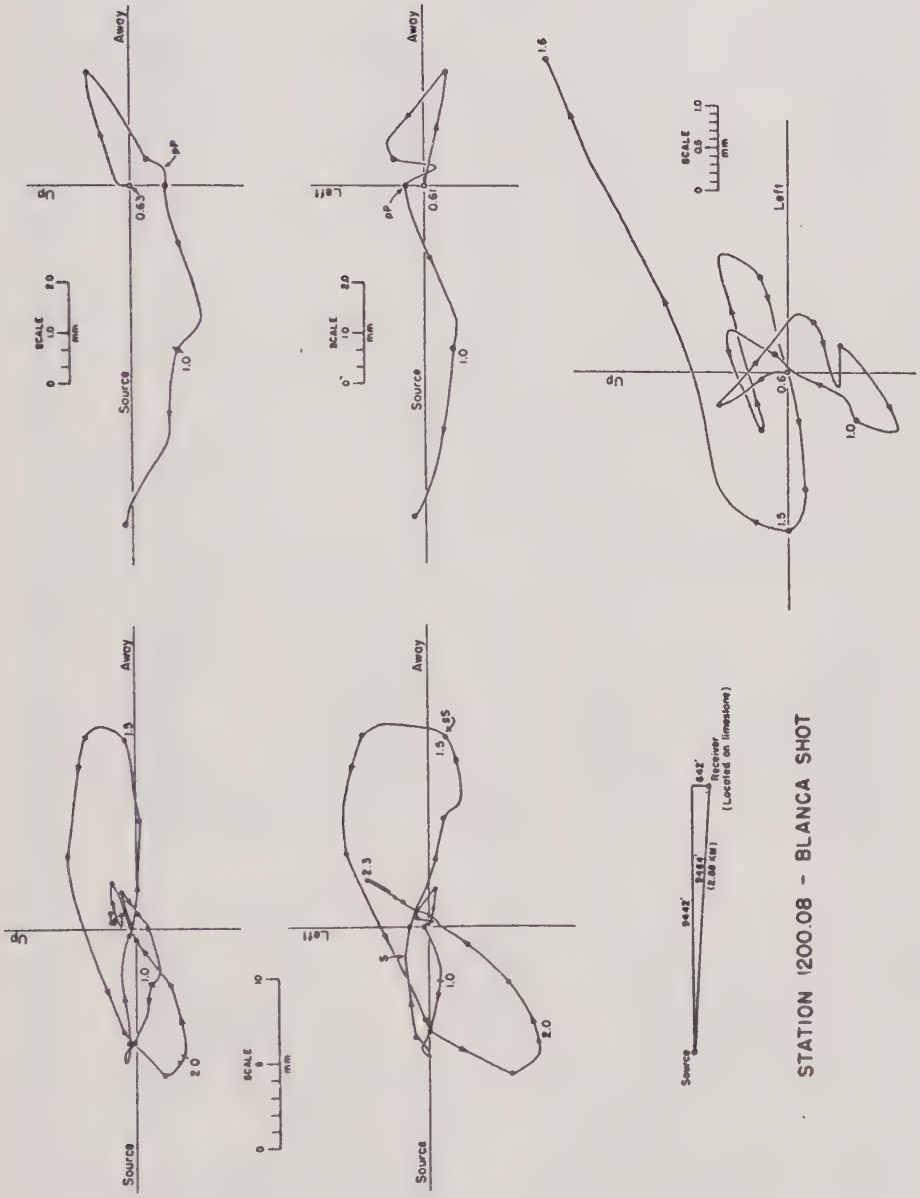
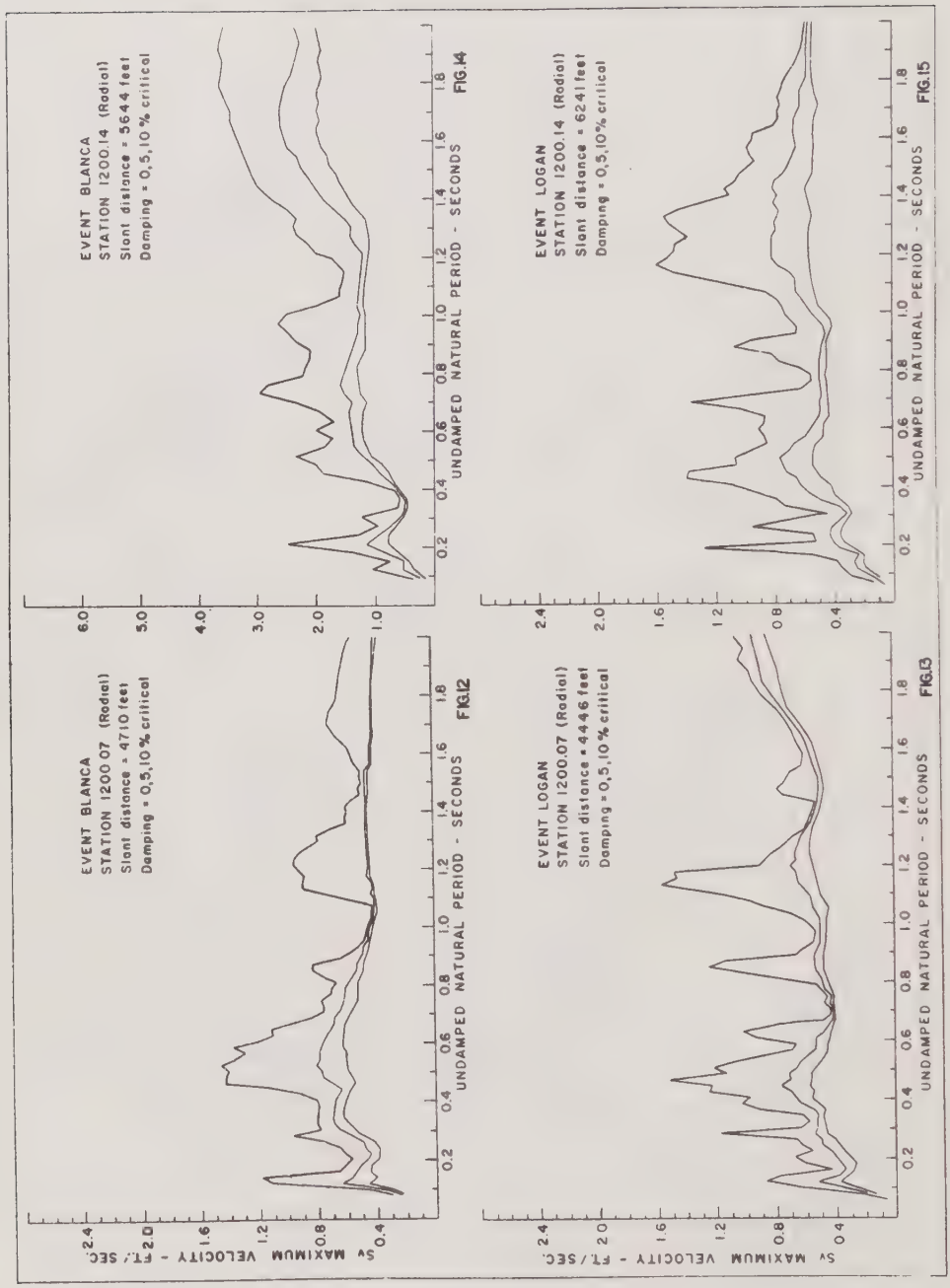
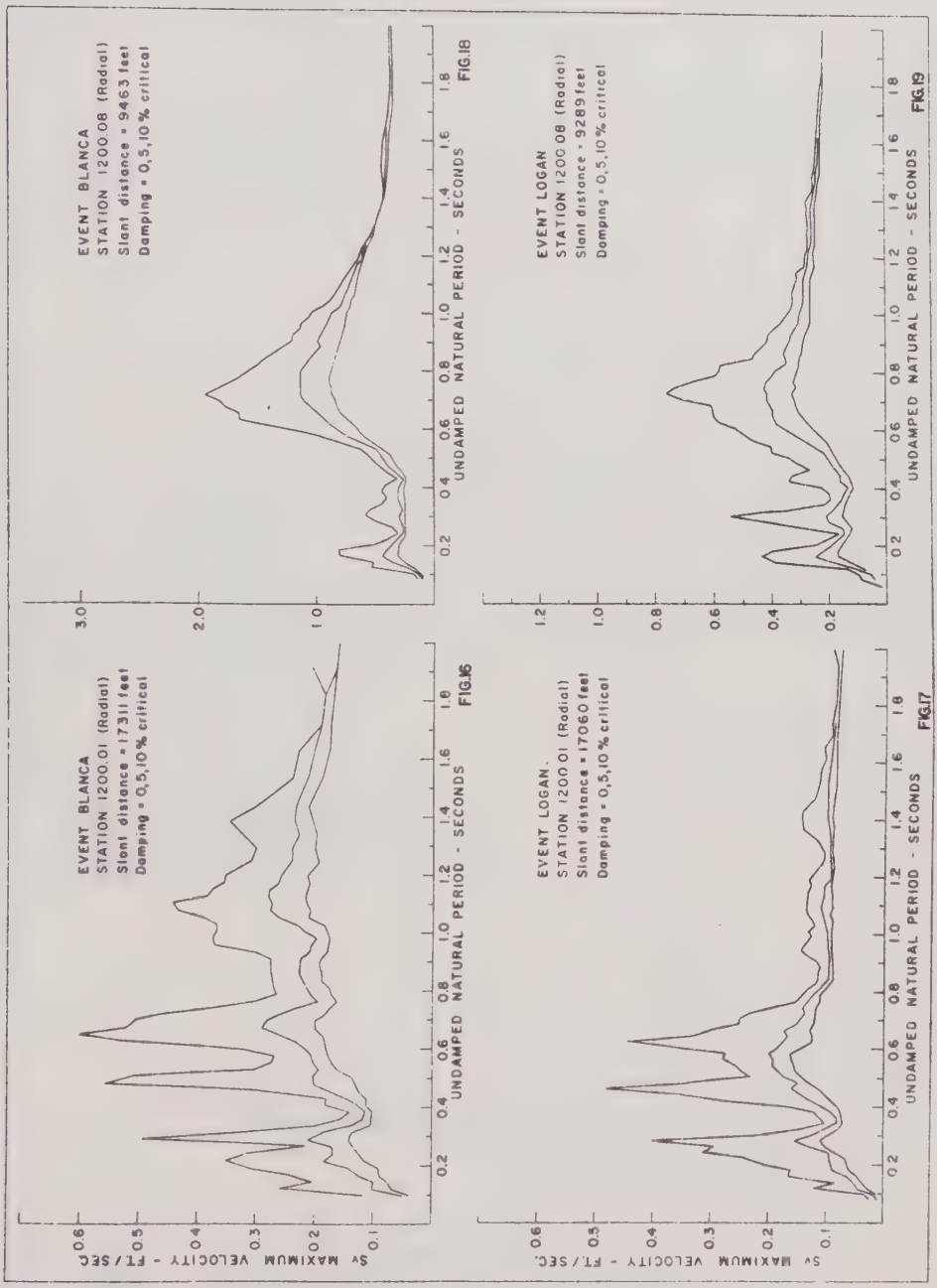


Figure 11--Trajectory of particle motion - Blanca Shot. Station 1200.08


EVENT BLANCA
STATION 1200.07 (Radial)
Slant distance = 4710 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
UNDAMPED NATURAL PERIOD - SECONDS
FIG.12
EVENT BLANCA
STATION 1200.14 (Radial)
Slant distance = 5644 feet
Damping = 0,5,10 % critical
UNDAMPED NATURAL PERIOD - SECONDS
FIG.14
EVENT LOGAN
STATION 1200.07 (Radial)
Slant distance = 4446 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
UNDAMPED NATURAL PERIOD - SECONDS
FIG.13
EVENT LOGAN
STATION 1200.14 (Radial)
Slant distance = 6241 feet
Damping = 0,5,10% critical
UNDAMPED NATURAL PERIOD - SECONDS
FIG.15


EVENT BLANCA
STATION 1200.01 (Radial)
Slant distance = 17311 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT/SEC.
UNDAMPED NATURAL PERIOD - SECONDS
FIG.16
EVENT BLANCA
STATION 1200.08 (Radial)
Slant distance = 9463 feet
Damping = 0,5,10 % critical
UNDAMPED NATURAL PERIOD - SECONDS
FIG.18
EVENT LOGAN.
STATION 1200.01 (Radial)
Slant distance = 17060 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT/SEC.
UNDAMPED NATURAL PERIOD - SECONDS
FIG.17
EVENT LOGAN
STATION 1200.08 (Radial)
Slant distance = 9289 feet
Damping = 0,5,10 % critical
UNDAMPED NATURAL PERIOD - SECONDS
FIG.19

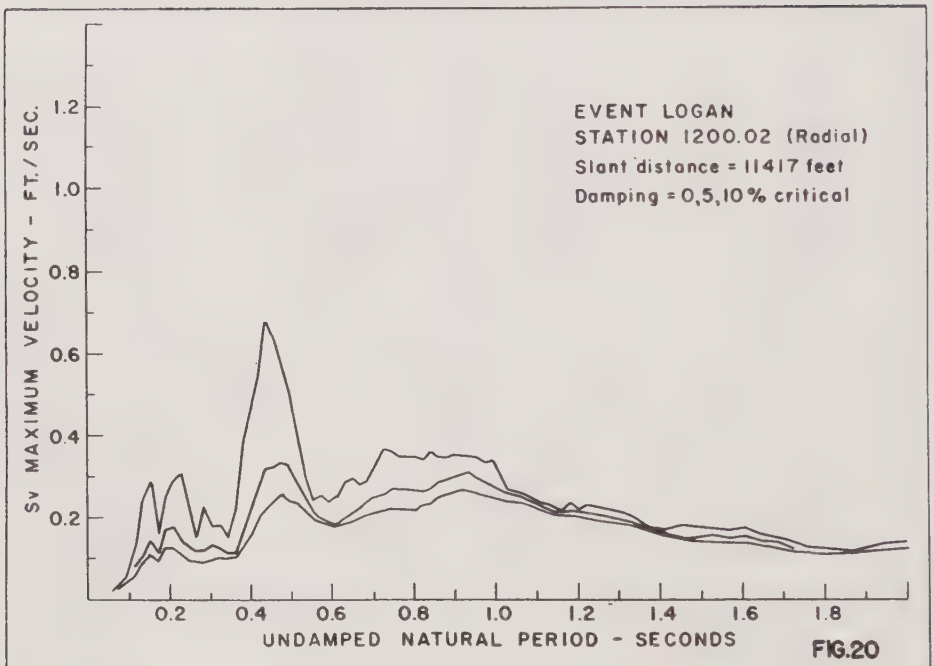

EVENT LOGAN
STATION 1200.02 (Radial)
Slant distance = 11417 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
1.2
1.0
0.8
0.6
0.4
0.2
0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8
UNDAMPED NATURAL PERIOD - SECONDS

FIG.20

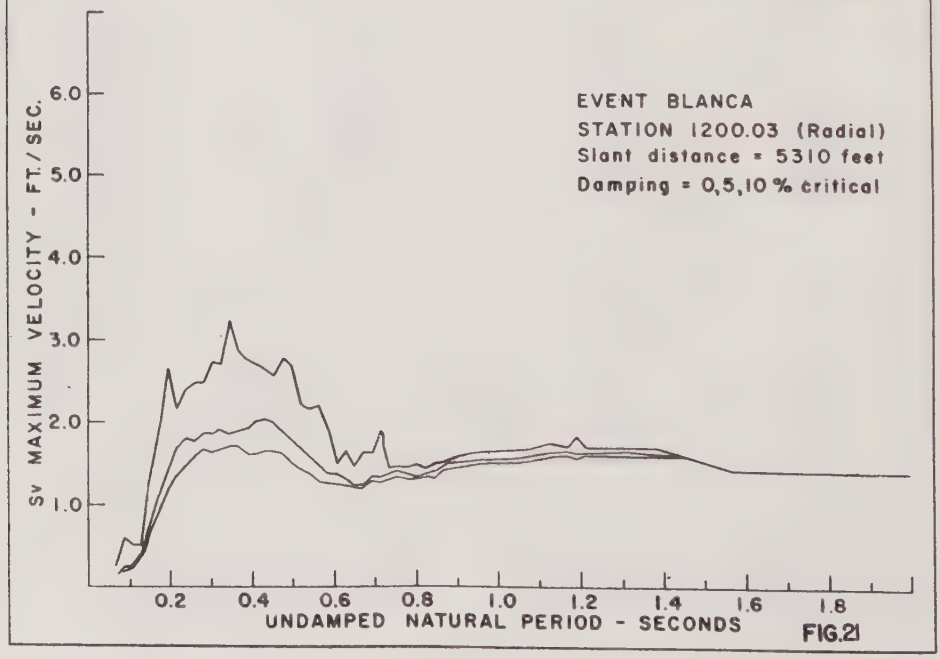
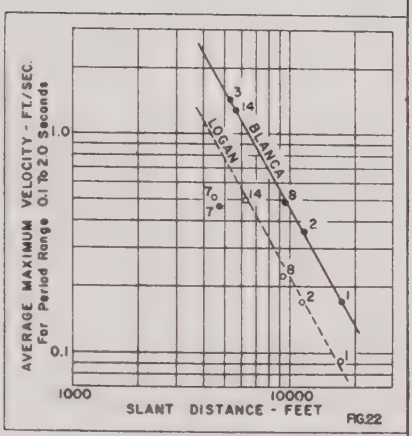
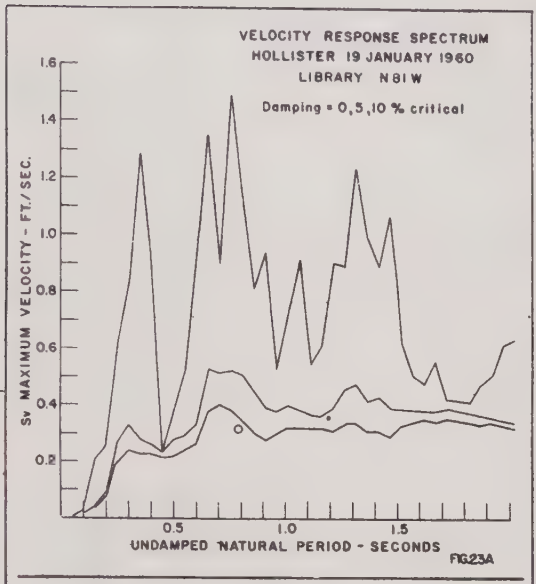

EVENT BLANCA
STATION 1200.03 (Radial)
Slant distance = 5310 feet
Damping = 0,5,10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
6.0
5.0
4.0
3.0
2.0
1.0
0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8
UNDAMPED NATURAL PERIOD - SECONDS

FIG.21


AVERAGE MAXIMUM VELOCITY - FT./SEC.
For Period Range 0.1 To 2.0 Seconds
1.0
0.1
1000
10000
SLANT DISTANCE - FEET
LOGAN
BLANCA
3
14
8
2
1
FIG.22


VELOCITY RESPONSE SPECTRUM
HOLLISTER 19 JANUARY 1960
LIBRARY N81W
Damping = 0, 5, 10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.5
1.0
1.5
UNDAMPED NATURAL PERIOD - SECONDS
FIG.23A

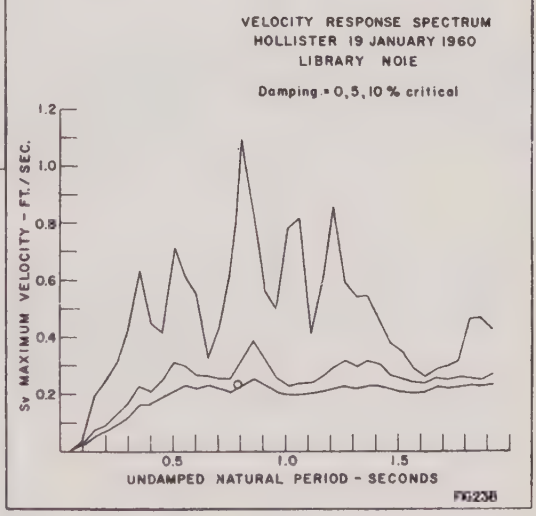

VELOCITY RESPONSE SPECTRUM
HOLLISTER 19 JANUARY 1960
LIBRARY N01E
Damping = 0, 5, 10 % critical
Sv MAXIMUM VELOCITY - FT./SEC.
1.2
1.0
0.8
0.6
0.4
0.2
0.5
1.0
1.5
UNDAMPED NATURAL PERIOD - SECONDS
FIG.23B

TABLE 1. Data on underground nuclear detonations.

Event	Date	Time (UT)	N. Latitude	W. Longitude	Elevation (ft)	Yield (kt)
Rainier	19 Sept 1957	1659:59.454	37 11 44.800	116 12 11.35	6615	1.7 ± 0.1
Tamalpais	8 Oct 1958	2200:00.131	37 11 43.10	116 12 01.64	6616	$0.070 \pm .010$
Logan	16 Oct 1958	0600:00.140	37 11 03.03	116 12 04.04	6141	$5.0^{+0.2}_{-0.4}$
Evans	29 Oct 1958	0000:00.15	37 11 41.46	116 12 17.03	6620	$0.055 \pm .030$
Blanca	30 Oct 1958	1500:00.15	37 11 09.36	116 12 07.28	6138	19.0 ± 1.5

TABLE 2. Comparison of seismic energy calculated by use of tentative formula with seismic energy calculated by seismologists.

	V (1) (ft/sec)	D (2) (ft)	E From Formula (3) (logarithm - ergs)	E From Seismologists (4) (logarithm - ergs)
Logan Nuclear Detonation	0.210	10,000	18.1	18.0 - 18.1 (Carder)
Hollister, California, Earthquake of 19 January 1960				
	0.285	25,800	18.9	18.8 (Tocher)
Hebgen Lake, Montana, Earthquake of 17 August 1959				
Bozeman, Mont.	0.254	305,000	20.8	21.9 (Richter)
Port Hueneme, California, Earthquake of 18 March 1957				
Port Hueneme	0.80	26,000	19.4	18.3 (Richter)
San Francisco, California, Earthquake of 22 March 1957				
Golden Gate Park	0.284	32,200	19.1	
State Building	0.310	44,300	19.4	19.2 (Tocher)
Alexander Building	0.140	50.700	19.2	
Oakland City Hall	0.104	86,600	19.4	

(1) For Logan, "V" was obtained from radial component spectrum; for the earthquakes, "V" was obtained from the largest horizontal direction spectrum.

(2) For Logan, "D" is slant distance to detonation; for earthquakes, "D" is horizontal distance to epicenter.

(3) Formula: $E = \frac{10^{12} VD^{1.78}}{2.4}$

(4) For earthquakes, seismic energy was calculated from magnitude rating of seismologist using Gutenberg-Richter formula:

$$\text{Log } E = 9.4 + 2.14\ M - 0.054\ M^2$$

DISCUSSION

C. F. Richter, Visiting Professor at the University of Tokyo, Japan:

I believe the magnitude value used for the Montana earthquake of August 1959 was taken from the Pasadena preliminary bulletin. If so, it represents a preliminary estimate based on recordings at the Pasadena station only, and subject to revision.

W. K. Cloud:

Preliminary estimate of magnitude was used. To my knowledge no other magnitude rating by the seismological laboratory has been published to date.

SEISMOLOGICAL STUDY OF THE CRUSTAL LAYERS IN INDIAN REGION FROM THE DATA OF NEAR EARTHQUAKES

By

K. C. Chakravortty and D. P. Ghosh*

Introduction

Seismologists investigated the various aspects of crustal structures in different parts of the earth from a study of near earthquakes and other methods. Those studies have revealed that the earth's crust in many regions consists of two or three layers of total thickness of 30 to 50 Kms, each layer being associated with characteristic velocities of P and S waves. Jeffreys on the basis of the data of European earthquakes deduced velocities in the granitic layer, the intermediate layer and the material just below as follows:

P_g = 5.57 ± .02 km/sec.	S_g = 3.36 ± .01 km/sec.
P* = 6.50 ± .03 "	S* = 3.74 ± .03 "
P_n = 7.76 ± .03 "	S_n = 4.36 ± .02 "

He gave the P and S velocities in an overlying sedimentary layer in European region as 4.7 and 2.8 km/sec. respectively. Gutenberg's study of Southern Californean earthquakes led to the velocities in different layers as:

P_g = 5.58 ± .01 km/sec.	S_g = 3.26 km/sec.
P_y = 6.05 km/sec.	S_y = 3.65 "
P_m = 6.95 ± .18 km/sec.	S_m = 4.10 "
P_n = 8.06 ± .11 "	S_n = 4.45 "

His analysis for regions under and near the Alps gave the following results regarding the thickness of different layers and the depth of Mahorovicic discontinuity:

Region	Thickness (km)		Depth of Mohorovicic discontinuity (km)
	Upper granitic layer	Intermediate layer	
North Western Europe	30	10	40
Schwefische Alb	25-30	20-25	50
Northern Alps	35	20-25	55-60

*Department of Geophysics, Banaras Hindu University.

Southern Carnic Alps	40	20-25	60-65
Yugoslavia	15	25	40

Rozova and Matuzawa arrived at the undermentioned figures in respect of Central Asia and Japan respectively:

	Velocity in km	
	Central Asia	Japan
P_g	5.54	5.0
P_*	5.99	6.20
P_n	7.82	7.5
S_g	3.29	3.15
S_*	3.79	3.7
S_n	-	4.5

Rozova deduced a thickness of 20-25 kms for the first layer (granitic) and a depth 45 kms for the Mohorovicic discontinuity in Central Asia. Roy on his study of Great Bihar earthquake of 15. 1. 1934 (Epicentre 26°.18' N 86°.18' E) gave the following velocities of P and S waves for Gangetic valley (India)

P_g = 7.80 km/sec.	S_g = 4.38 km/sec.
P_* = 6.21 "	S_* = 3.71 "
P_n = 5.26 "	S_n = 3.29 "

His calculations of thickness of granitic layer and basaltic layer in Bihar area were 14.8 kms and 25.4 kms respectively. Banerjee on an examination of the after shocks of the Great Assam earthquake of 15. 8. 1950 indicated that the depth of the Mohorovicic discontinuity under the foot hills of the Himalayas is some what greater than 60 kms. Chakravorty in his Presidential Address, Mathematics Section, Indian Science Congress expressed the view that the depth of the Mohorovicic discontinuity under the Himalayas might be of the order of 70 kms. It is seen from the results described above that the crustal aspects of different parts of the earth are far from being identical. Depths of crustal layers and the wave velocities in the layers vary from one region to the other. One of the authors of this paper in his routine study of Seismograms relating to near shocks in India noticed at times that some of the phases in the Seismograms did not tally well with the standard Travel Time tables which are based on average values of wave velocities. With due regard to the personal errors and errors in time determinations which may be responsible for the anomaly observed, one has to examine critically whether the discrepancies are real and if so, whether it is

possible to explain them. Apart from this question, it is interesting to examine how the crustal aspects in Indian areas compare themselves against those of other areas which were studied from extensive data. It has, therefore, been considered worthwhile to make an attempt to find out the velocities of P and S waves in different crustal layers and the thickness of these layers from a study of a large number of near earthquakes of shallow focal depths recorded at Indian Stations and to find out the actual focal depths of the earthquakes. The object of this paper is to describe the results obtained from this study and to discuss the interesting features.

Selection of data

For the purpose of the present analysis 19 near earthquakes of shallow focal depths recorded by the Seismological observatories in India during the period June 1950 to February 1954 were selected on the following considerations (vide Tables 1 & 2):

(a) The epicentres of these earthquakes were determined by the India Metereological Deptt. from a study of all available observations and the relevant data were published in the Seismological Bulletins. Many of the selected earthquakes were felt.

(b) Most of the desired phases (viz. P_g, P_*, P_n, S_g, S_*, S_n) in these cases were available from the records of at least two stations.

(c) The epicentres in question were fairly well distributed over Indian area and its immediate neighbourhood.

In regard to near shocks Macalwane pointed out that a phase is real if it can be followed from station to station in such wise that the respective arrival times when plotted against epicentral distances are isolated and lie on or very close to a continuous curve. Due consideration was given to this point in selecting out the above data from a large number of earthquake records.

Analysis of data

Seismologists have accepted the view that in the regions close to the epicentre i.e. when the epicentral distances from the recording stations are within 10 degrees or so, the additional pulses P_*, P_g, S_* and S_g are due to waves which travel through layers near the earth's surface. If these pulses along with normal P_n and normal S_n are recognised on the Seismograms of a few near stations and if the epicentre of the earthquake and its "0" time are determined correctly, it is possible to find out the velocities of P_g, P_* etc. in different layers and the corresponding delay times, from which the focal depth of the earthquake as well as the thickness of the individual layers can be calculated from the following formulae due to Jeffreys:

$$d = \frac{\beta_g}{(1 - \frac{\beta_g^2}{\alpha_g^2})^{\frac{1}{2}}} . t_{P_g - S_g},$$

$$2 H_g = d + \frac{\alpha_g}{(1 - \frac{\alpha_g^2}{\alpha_*^2})^{\frac{1}{2}}} \cdot t_{P_* - P_g},$$

$$2 H_B = d + \frac{\alpha_*}{(1 - \frac{\alpha_*^2}{\alpha^2})^{\frac{1}{2}}} \cdot t_{P - P_*}$$

when α_g, α_* and α are the velocities of P waves in different layers; β_g, β_* and β are the velocities of S waves in different layers;

$t_{P_g - S_g}$, $t_{P_* - P_g}$; $t_{P - P_*}$ are the delay times;

d is the depth of focus

H_g is the thickness of the first layer (granitic)

H_B is the thickness of the second layer (basaltic)

In the present investigation, the time data for all available phases in respect of each earthquake were treated in a way similar to Geiger's method of eliminating the errors in order to find out the correct "O" time and the epicentral distance. After this treatment final time distance curves were drawn and the corresponding delay times were calculated. The Sample Curves drawn in respect of one earthquake (29. 8. '53) are shown in Figure 1. The delay times calculated from the curves are also shown as inset in this figure. From the time distance curves thus drawn for each of the 19 earthquakes, the depth of focus, velocities of the different types of waves characteristic to individual layers and the thickness of the layers were calculated. The results are given in Tables 1 & 2 attached. The values of velocities and layer thickness shown in the Tables are, no doubt, appropriate to the regions covered by a distance of 1000 kms. or so round each epicentre. The locations of the Seismological stations which recorded the earthquakes and the epicentres of all the 19 earthquakes studied are shown in Figure 2. In this figure the layer thicknesses obtained from the data relating to each earthquake are also shown for ready reference.

Discussion on the results

It will be seen from Table 1 that the depths of foci of almost all the earthquakes under consideration are below 20 kms and that the foci lie either near the bottom of the first layer (granitic) or at the top of the second layer (basaltic). The latter characteristic is exhibited particularly by the earthquakes whose epicentres are in Tibet. Roy identified the focus of the Great Bihar earthquake of 15. 1. 1934 at the bottom of the granitic layer. Banerjee discussed the mechanism of the shallow-focus earthquakes in India. According to him materials at a depth of 20 kms or more apparently exist in a state of cohesion under the confining pressure and in producing earthquakes of the type under examination, nature breaks a block of large dimension (say 200 kms long, 100 km

broad and 10 kms thick) and not a large no. of rocks. If the upper surface of this block be 15 or 20 kms below the surface, the material will be under high pressure and high temperature. The depths of foci obtained from the present analysis are quite consistent with the idea put forward by Banerjee. It is interesting that the depths of foci for epicentres to the north of Lat. 28° N in India and neighbourhood are in some cases slightly less than those to the south. The enhanced pressure under the Himalayan region may be responsible for this feature. Apparent existence of foci at the top of the second layer in the cases discussed is interesting in the light of Banerjee's discussion regarding the mechanism of shallow focus earthquakes, because it will be noticed from the note which will follow that the thickness of the first layer in the areas where the foci lie at the top of the second layer is comparatively small. It is to be mentioned here that the existence of foci at the top of the second layer does not materially affect the time distance curves used for the calculation of thickness etc. of different layers.

According to Jeffreys the "P" usually observed in a near earthquake is a wave which originated from the focus as a shear wave and was transformed to a "P" wave at incidence on the surface and travelled along the surface to the station. Byerly, however, recognised both the secondary P of Jeffreys and also a weaker direct P_g. Gutenberg opined that P_g travels directly from the focus to the station and P_g on incidence with the surface sets up a shear surface wave which travels faster than S_g. While it was difficult during the present study to verify whether the Indian earthquakes revealed anyphases other than P_g, P_*, P_n, S_g, S_* & S_n, it was confirmed from all the travel time curves that S_g cuts the time axis at the epicentre earlier than other pulses.

The calculated values of thickness of layers as given in Table 1 and shown in Fig. 2 present an interesting picture. While the thickness of the first layer (granitic) varies between 13 and 20 kms, it shows a characteristic change from one region to another. To the north of Lat. 30° N the thickness is 13 or 14 kms; it increases southwards to 20 kms. The thickness of the second layer (basaltic) is 17 to 19 kms in southern part of Central Himalayan region and 22 to 27 kms in Assam Kutch, Arakan and the rest of Himalayan region. The values of total thickness for the two layers gives an uneven picture of the depth of the Mohorvicic discontinuity.

The total thickness varies between 35 and 42 kms, the southern parts of Central Himalayan region showing slightly smaller values than other regions. The maximum crustal depth occurs over Eastern Himalayan region. The values thus obtained fixes the Mohorvicic discontinuity at a depth not exceeding 42 kms. While this result agrees fairly well with that obtained by Roy in his study of Great Bihar Earthquake, it is difficult to conceive a depth of 60 or 70 kms of the discontinuity as suggested by some workers. Airy's theory of isostasy requires that the Himalayas should have deep roots.

The results obtained from the present analysis do not in any way go against the above requirement because a difference of 7 kms in the total crustal thickness from one region to the other is revealed by this

analysis. In this connection one should not lose sight of the fact that the elevation of the Himalayan region varies considerably from one place to the other. Gutenberg's value for depth of Mohorvicic discontinuity under and near the Alps exceeds 50 kms. The question, therefore, remains to be examined critically how the Himalayan region, at least at places, shows a less depth for the discontinuity.

In regard to the velocities for the different types of waves it will be seen from Table 2 that the values vary widely from one region to the other. S waves being slower show smaller variation than P waves. Then again the amplitudes of variations of other waves are less prominent than those of P_g and P_*. It is fairly clear that the physical properties of crustal layer near the surface are widely different, and this difference is responsible for varying velocities. It is interesting that velocities in lower Latitudes are generally less than those in the Himalayan region. Such variations in velocities were observed in other parts of the earth by many Seismologists.

In different parts of Asia itself the velocities were found to vary considerably, for P_g 3.56 to 5.54 km/sec., for P_* 5.99 to 6.30 km/sec., for P_n 7.5 to 8.48 km/sec., for S_g 2.22 to 3.29 km/sec., for S_* 3.7 to 3.79 km/sec. and for S_n 4.4 to 4.8 km/sec. In the light of the above findings the variation of velocities in different parts of Indian areas is not unusual.

Conclusion

In view of the varying aspects of the crustal layers and the respective velocities in the Indian areas as discussed in the preceding paragraphs one can appreciate the difficulty which a Seismologist is likely to experience to find out the correct epicentral distances etc. of near shocks in India on the basis of the standard Travel Time Tables computed from extensive data collected from all parts of the earth. If it is possible to work out Travel Time Tables on the basis of regional data over India and neighbourhood the task of analysing the near earthquakes becomes easier. It is quite probable that the discrepancies in the determination of correct phases of near earthquakes as pointed out before, lie to some extent in the lack of information regarding the correct parameters of the waves travelling in the crustal layers in Indian areas.

One of the authors had the opportunity of having a short discussion on this point with Sir Harold Jaffreys during his visit to Banaras Hindu University in November, 1959. Sir Jaffreys apparently realised the difficulty discussed above. In recent years much information on the structure of the outermost layers in the Indian region has been obtained by reflection and refraction methods of seismic prospecting besides from a large volume of data relating to near earthquakes. It is hoped that the question of finding out correct parameters for the Seismic waves in Indian region will get proper consideration by the Indian Seismologists in due course. The findings from the present study may be helpful in that connection.

Acknowledgement

The authors are thankful to the India Meteorological Department and particularly Dr. A. N. Tandon, the Chief Seismologist of that Department, for their valuable co-operation in making the Seismological data available to Banaras Hindu University. Thanks are also due to Dr. Rajnath, Head of the Department of Geophysics, Banaras Hindu University for the facilities he extended to the authors during this work.

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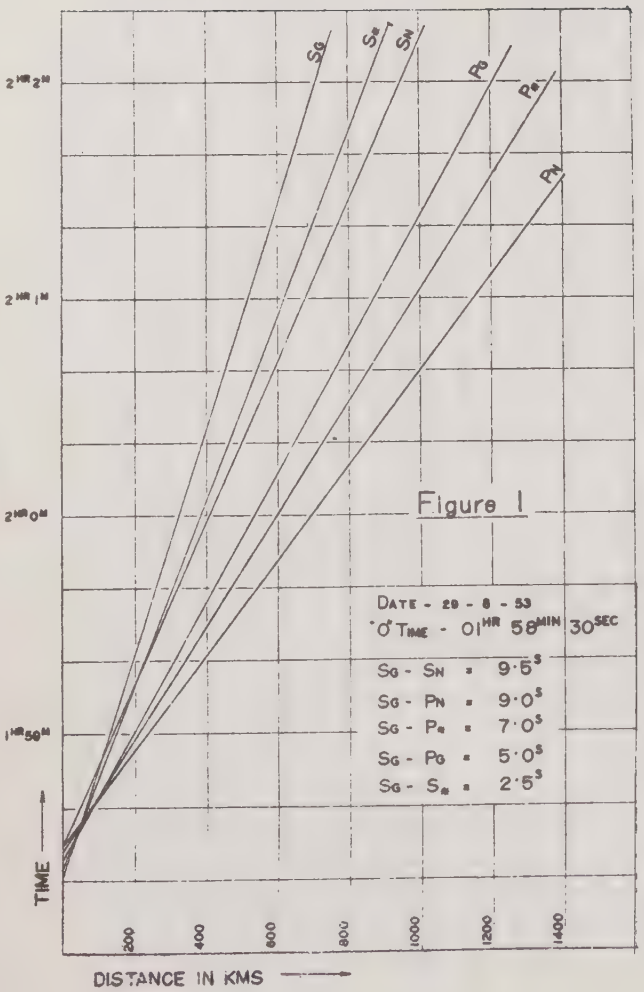

Figure 1

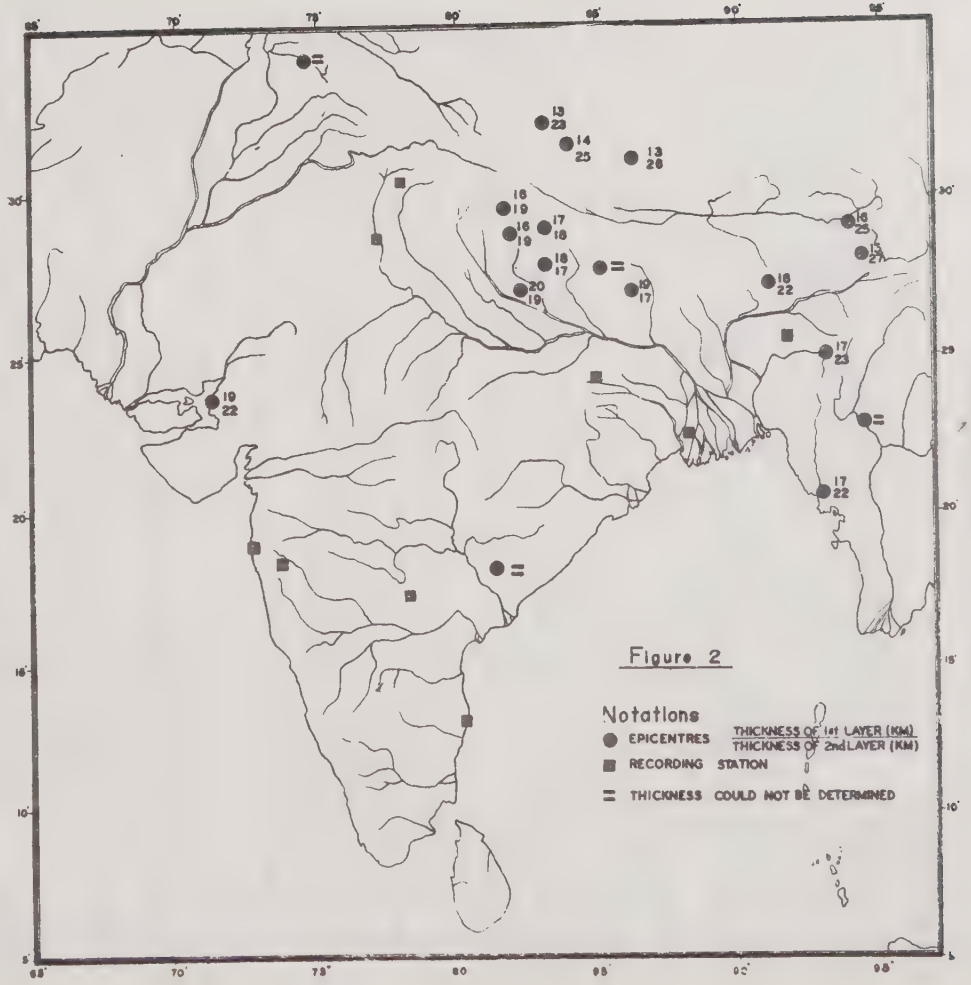
Notations
EPICENTRES
THICKNESS OF 1st LAYER (KM) / THICKNESS OF 2nd LAYER (KM)
RECORDING STATION
= THICKNESS COULD NOT BE DETERMINED

Figure 2

Table 1

Epicentres and focal depths of earthquakes and thickness of crustal layers

S.No.	Date	Epicentre		Focal depth	Thickness in km		
		Lat (N)	Long(E)	in km	1st layer	2nd layer	Total
1.	14.6.50	24.0	71.2	19	19	22	41
2.	14.4.51	29.0	94.0	16	16	25	41
3.	22.4.51	28.7	94.4	16	15	27	42
4.	28.5.51	27.6	85.0	-	-	-	-
5.	9.7.51	20.5	93.0	16	17	22	39
6.	16.1.53	25.0	93.0	17	17	23	40
7.	27.1.53	23.0	94.5	-	-	-	-
8.	16.2.53	29.5	81.5	16	16	19	35
9.	20.2.53	27.0	86.0	21	19	17	36
10.	23.2.53	28.7	81.8	14	16	19	35
11.	27.2.53	29.0	83.0	16	17	18	35
12.	1.3.53	34.0	74.5	-	-	-	-
13.	23.4.53	28.0	83.0	16	18	17	35
14.	29.8.53	27.3	82.2	19	20	19	39
15.	8.10.53	31.5	84.0	16	14	25	39
16.	11.10.53	32.0	83.0	14	13	23	36
17.	3.12.53	31.0	86.0	16	13	26	39
18.	5.1.54	18.0	81.3	-	-	-	-
19.	23.2.54	27.0	91.0	18	18	22	40

Table 2

Velocities of P and S waves

S.No.	Date	Epicentres of earthquakes		Velocities in km/sec. for					
		Lat (N)	Long(E)	P_g	P_*	P_n	S_g	S_*	S_n
1.	14.6.50	24.0	71.2	5.5	6.4	7.9	3.5	3.9	4.5
2.	14.4.51	29.0	94.0	5.1	6.3	7.7	3.4	3.8	4.3
3.	22.4.51	28.7	94.4	5.2	6.2	7.7	3.3	3.8	4.4
4.	28.5.51	27.6	85.0	-	-	7.8	3.5	-	4.5
5.	9.7.51	20.5	93.0	5.5	6.5	7.8	3.5	3.9	4.5
6.	16.1.53	25.0	93.0	5.5	6.4	7.8	3.3	3.8	4.5
7.	27.1.53	23.0	94.5	-	6.4	7.8	-	3.8	4.4
8.	16.2.53	29.5	81.5	5.3	6.3	7.7	3.3	3.7	4.3
9.	20.2.53	27.0	86.0	5.3	6.2	7.6	3.3	3.7	4.3
10.	23.2.53	28.7	81.8	5.2	6.2	7.6	3.3	3.7	4.4
11.	27.2.53	29.0	83.0	5.1	6.3	7.6	3.3	3.7	4.4
12.	1.3.53	34.0	74.5	-	6.4	7.7	-	3.8	4.4
13.	23.4.53	28.0	83.0	5.4	6.4	7.8	3.2	3.7	4.4
14.	29.8.53	27.3	82.2	5.6	6.5	7.7	3.2	3.9	4.4
15.	8.10.53	31.5	84.0	5.1	6.2	7.6	3.2	3.8	4.4
16.	11.10.53	32.0	83.0	5.2	6.3	7.6	3.2	3.7	4.3
17.	3.12.53	31.0	86.0	5.3	6.4	7.6	3.3	3.8	4.3
18.	5.1.54	18.0	81.3	-	-	7.9		-	4.5
19.	23.2.54	27.0	91.0	5.5	6.5	7.8	3.4	-	4.5

SEISMICITY OF THE SOUTH OF THE KURIL ISLANDS

By S. A. Fedotov*

Thorough seismic investigations show that earthquakes of over 9 degrees are unlikely to occur on the territory of the Southern Kuril Islands.

In 1957-1959 the Institute of the Earth's Physics of the USSR Academy of Sciences was conducting, under the IGY programme, complex geophysical and geological investigations on the territory of the Kuril Islands, Kamchatka, Okhotsk Sea and Kuril deep-water trough the aim of which was to study the area of transition from the continent to the ocean (I).

This work planned a thorough investigation of seismicity, its relations with tectonics and volcanism, construction of vertical velocity sections of the Earth's crust and the mantle's upper strata, the study of the spectral and energy characteristics of earthquakes of the region, insertion of more accurate data into the map of seismic zoning on the Kuril Islands. To carry out the seismic investigations, a network of high-sensitive seismic stations has been organized in 1957 and they continue the observations in 1960 too. Their location is given in Fig. 1.

The temporary seismic stations of the Southern Kuril network have equipment that is now usually used by the Soviet scientists conducting seismic observations.

To record the three components of the ground oscillations electrodynamical vibrographs VEGIK and galvanometers GB-IY are used. The period of own oscillations of the VEGIK pendulum is easily regulated within 0.5-1.5 sec. The frequency of own oscillations of the suspension system of the GB-IY galvanometer is about 14 cycles. The description of these devices is given in paper (2). The registering channels of these stations have equal frequency characteristic (Fig. 2) which was selected after an experimental registration with different frequency characteristics. Cuts of the characteristics for low and high frequencies are necessary to neutralize storm micro seisms and wind disturbances the level of which is rather high on the Kuril Islands. Each station has four recording channels. Three channels of the station record components of the ground oscillations during weak earthquakes with maximum magnification 10.000. The fourth channel is assigned to record felt earthquakes. It has maximum magnification 500 which at a selected frequency characteristics ensures, within the limits of the seismogram, records of earthquakes with intensity up to 5 degrees.

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The registration speed is 240 mm/min as the result of which the errors in time readings on the seismogrammes do not exceed ± 0.15 sec.

Among the supplementary equipment mention should be made of a device for automatic increase of the brightness of light rays during an earthquake, a device for an automatic reception of precise time signals and circuits switching in at the moment of an earthquake to record it by an oscillograph at a speed of 600 mm/min and more for a subsequent analysis of the spectrum.

The above equipment records a great number of weak near earthquakes, the efficient time service helping it to reduce errors in determining the epicentres and their depths, the equipment obtains data on seismicity of the area under investigation within short periods of time.

Thorough seismic observations have confirmed those conceptions which have been formed up on the seismicity of the Southern Kuril Islands on the basis of the data of the permanent seismic stations (4), (5), (6) used as a cornerstone for seismic zoning in 1957 (3).

The Earth's crust within the limits of the Southern Kuril Islands possesses low seismic activity. No strong earthquakes occur there (4), (5), (6) and as evidence expeditionary observations not many weak earthquakes occur.

During January-October 1958 the Earth's crust in the zone of the Iturup Island saw less than 40 weak earthquakes which were distinctly recorded by all the temporary stations on the Iturup Island that make for 10 percent of the total number of weak near earthquakes having been recorded by these stations during the same span of time, Fig. 3. Thus, a compact group of weak earthquakes has been distinguished that are being recorded at the Reydovoe station with S-P = 0.1 - 0.9 sec. These earthquakes may be of volcanic origin.

The greater part of weak and strong earthquakes in the Southern Kuril zone in 1957-59 occurred under the Earth's crust between the islands and the deep-water through. An illustration of this is a map of near earthquakes for March 1959 which is a typical monthly map of epicentres from data of the temporary stations in the Southern Kuril zone obtained in 1957-59. In the same area there occurred a strong tsunami earthquake dated as far as November 6, 1958 (7).

Data on the strong Southern Kuril Islands for the last 50 years (4), (5), (6) and expeditionary observations of weak earthquakes permit to arrive at the following conolusions.

Strong surface earthquakes on the Southern Kuril Islands are unlikely. Dangerous for constructions in these regions are strong earthquakes the minimum hypocentral distances to which exceed 100 km. and, as a rule, vary from 120 to 200 km.

At the above hypocentral distances the earthquakes intensity does

not exceed, on the average, 9 degrees, even during disastrous earthquakes with magnitude $M \geqslant 8$, that is confirmed by the consequencies of the Kamchatka Earthquake on November 4, 1952 (8) and of the Iturup Earthquake on November 6, 1958 (7), Fig. 5.

Thus, earthquakes with intensity exceeding 9 degrees are unlikely to occur on the territory of the Southern Kuril Islands.

The Kuril Islands are marked on the Map of Seismic Zoning of the USSR as a 9-degree zone (3). The expeditionary observations of recent years have confirmed the estimate.

The investigation of seismicity on the Southern Kuril Islands by the temporary stations is continued in the following directions:

a) the study of the spectral characteristics of the earthquakes occurring in the Southern Kuril zone;

b) the study of the dependence of the number of the earthquakes on their energy, especially, that of foreshocks and aftershocks of the earthquake on November 6, 1958 with M (magnitude) $\geqslant 8$;

c) the study of the relation of the seismicity and the Earth's crust structure and the upper layers of the mantle on the given area of the island arc.

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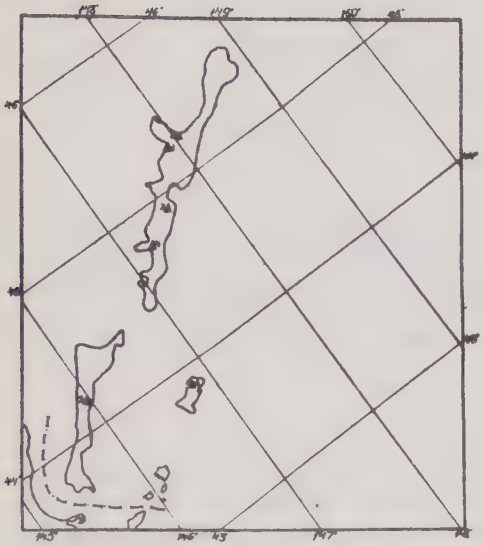

Fig. 1 The Southern-Kuril network of temporary stations.
1 - the permanent seismic station "Kurilsk",
2-6 - temporary stations,
2 - the temporary station "Reydovoe".

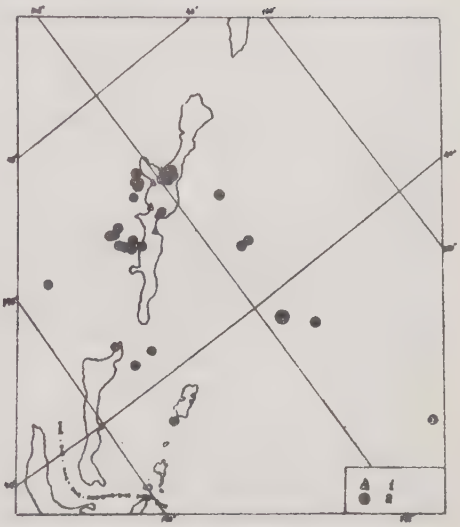

Fig. 3 The map of weak earthquakes in the Earth's crust of the Iturup zone for January-October 1958.
1 - temporary seismic stations that operated on the Iturup in 1958,
2 - earthquake epicentres in the Earth's crust.

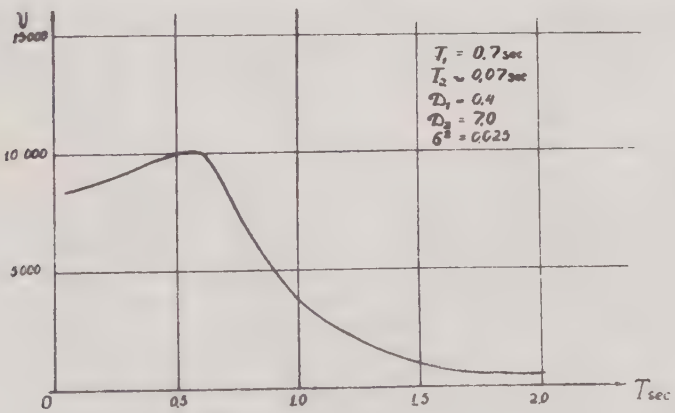

Fig. 2. Optimum frequency characteristics to record near earthquakes on the Kuril Islands.

V - magnification, T - period of ground oscillations,
T_1 and T_2 - periods of own oscillations of the vibrograph pendulum and of the galvanometer frame,
D_1 and D_2 - dampings of the vibrograph and galvanometer,
σ^2 - linking coefficient of the galvanometric registration.

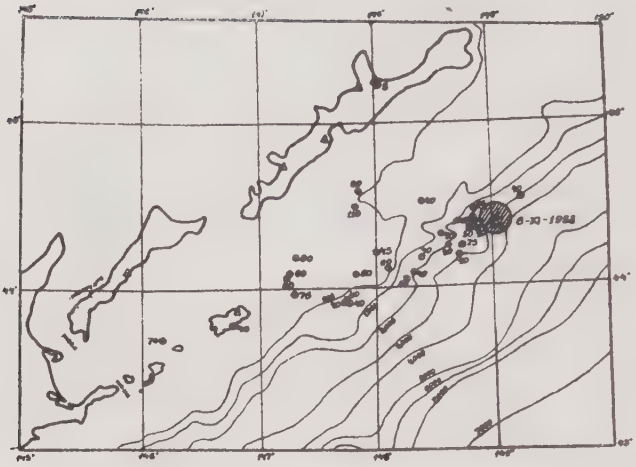

Fig. 4. The map of the epicentres of weak earthquakes of the Southern Kuril zone for March 1959. Next to the epicentres are indications of the earthquake depths in km.
The epicentre of the disastrous earthquake on November 6, 1958 is hatched. The isobates of the ocean are plotted.

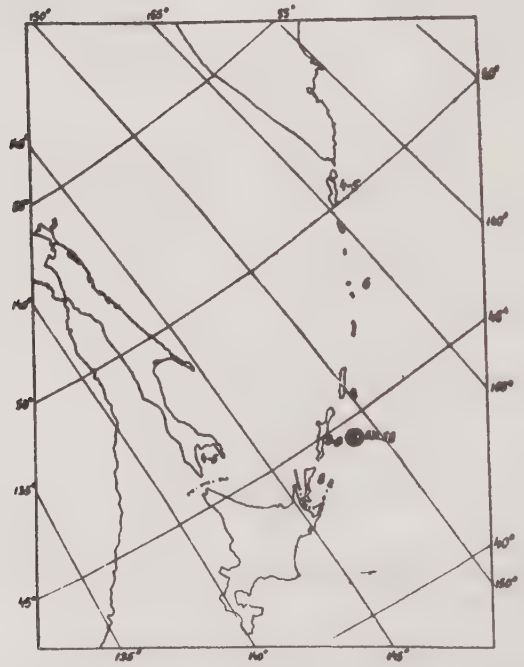

Fig. 5. The earthquake magnitude in degrees on November 6, 1958, the Kuril Islands.

Microseisms and Subsoil Conditions

By Fuyuhiko Kishinouye*

Abstract

Vibration characteristics of the ground by earth tremors were studied by K. Kanai, and applied to investigation of subsoil conditions and of earthquake-proof constructions. Similar relations were found about microseisms, or long period natural earth vibrations. Amplitude distribution of microseisms will show subsoil conditions in large area.

Introduction

The ground is always vibrating. The vibrations are so small in amplitude that usually no one can feel but seismographs of high magnification record them. The earth vibrations are called variously: earth pulsations, earth tremors, microseisms, microseismic disturbances and so on. Their period of vibrations are not constant. Traffics on road cause vibrations of 0.1 sec. period, those of 0.3 sec. period are observed in the uptown of Tokyo and of 0.6 sec. period in the downtown. These vibrations which are of shorter period than microseisms are studied by Dr. K. Kanai of our institute as "earth tremors" with his electric seismograph of which self-vibration period is 1 sec. He classified subsoil conditions in four categories with frequency distribution of vibration periods and with relation between amplitude and period. The classification is suggestive for design of earthquake-proof construction as it shows a dynamic property of subsoil.

The above-mentioned vibrations are of short period. Microseisms are longer in their period, so that they may provide a knowledge of subsoil in wider area than the earth tremors.

Studies on Microseisms

Microseism in Japan was noticed by J. Milne in 1883 when he was there. Many Japanese seismologists, of course, investigated microseisms. F. Omori found three periods of microseisms at Hongō in Tokyo: $q = 2.9$ sec., $Q_1 = 4.5$ sec. and $Q_2 = 7.5$ sec. Comparison of microseisms at different places, Hongō and Hitotsubashi, was also carried out by him. Then he concluded microseisms are proper vibrations of the ground.

Concerning the origin of microseisms, relation between microseisms and typhoon or tropical cyclone was studied. And microseisms were thought as surface waves of the earth caused by sea waves generated by violent wind of typhoon.

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In Europe microseisms were formerly considered to be an effect of breaking of rough sea waves on cliffs of coast, and are propagated in the continent. Recentry Longuet-Higgins published a new theory on the origin of microseisms. The observed data that periods of microseisms is a half of those of sea swells was noticed by him, and incorporated into his theory that microseisms are generated by standing sea waves due to incident and reflected waves at coast.

Investigation of microseisms as progressive waves was made by method of tripartite observations, or observations of microseismic waves at three stations simultaneously. Tripartite observations were carried out by the writer in 1927, and similarity of wave forms at different stations was found. The observations were interruped in Japan by the World War II, but the method was developed in Germany and was applied to weather forecasting in U.S.A.. The method was resumed in Japan soon after the war.

Accuracies of this observation were checked by simultaneous observations at four stations, or a pair of tripartite nets laid in close vicinity. In other cases, moreover, tripartite observations of different component observation was carried out at one of the tripartite stations.

After these critical studies, propagation velocity of microseismic waves was obtained in Tokyo. The velocity observed was about 1000 m/sec or less. Comparing the value with those in other countries, the velocity is small, nevertheless, amplitude of microseisms is large. From these results, it is suggested that amplitude of microseisms is larger while period is longer on soft subsoil than on hard ground.

Microseisms and Subsoil Conditions in Earthquake Districts

I. Shida who studied microseisms in Yamagata Prefecture with the writer showed relative amplitude of microseisms on Shōnai Plain. In Fig. 1 lines indicate equal amplitudes relative to microseisms observed at Sakata Meteorological Station. The lines have correlation to subsurface topography, in other words, microseisms are larger the thicker the alluvial deposite at the observation sites. On sand dunes along the coast of Japan Sea, amplitude of microseisms was unexpectedly small. The amplitude distribution of microseisms is closely related to the damage of the earthquake of 1894.

Correlation of amplitude of microseisms and earthquake damage is studied also in Ogasa County, Shizuoka Prefecture, where the Tōnankai earthquake was devastating. Tripartite observations were applied to study the effects of ground on damage of the earthquake. For the observations three electric seismographs designed by the writer for recording microseisms were used. The results thereof gave the same conclusion that there exists a positive correlation between earthquake damage and amplitude of microseisms.

The study of subsoil condition by observations of microseisms is still premature, but it will be useful for earthquake engineering.

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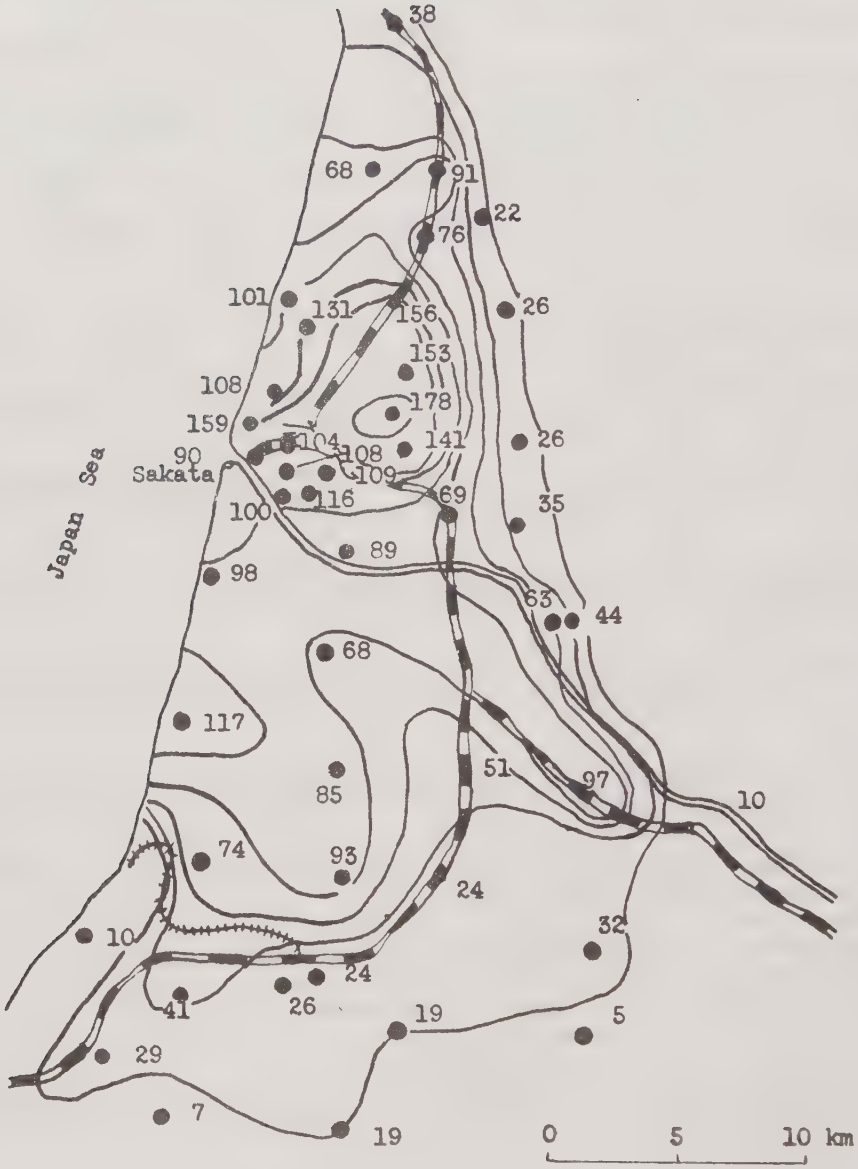

Fig. 1. Relative ampitude of microseisms in Shōnai Plain, Yamagata Prefecture.

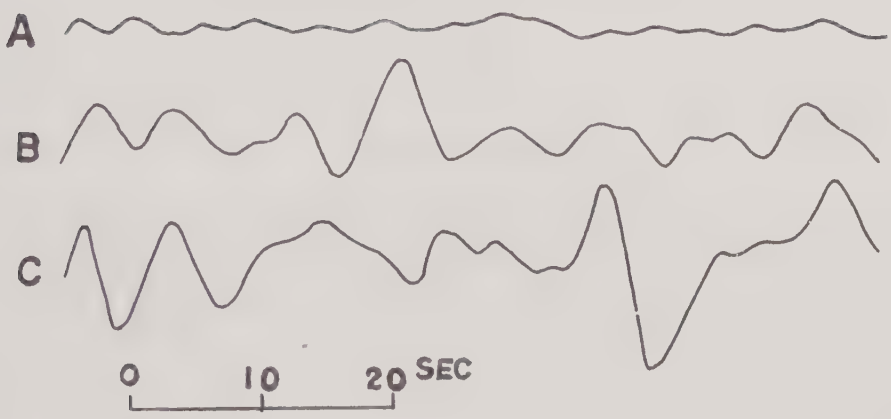

Fig. 2a. Tripartite observations at Ogasa, Shizuoka.
March 26, 1959.
Record in original size.

Fig. 2b. Positions of the tripartite stations recorded the above Seismograms.

GEOTECTONIC MOVEMENTS AND THEIR INFLUENCE ON THE HYDROGRAPHY OF INDO-GANGETIC PLAINS*

By

P. KUMAR**

INTRODUCTION

The seismic phenomena in relation to the geotectonics and structure of Extra Peninsula and occasionally of Indo gangetic plains have been dealt with by various authors such as Oldham (21), de Ballore (5) Dunn (7) Wadia (2729) Auden (3 & 4). Further various accounts of the changes in the pattern of hydrography and paleogeography of the Indo gangetic plains in Recent era have been published by Oldham (17), Pascoe (22), Pilgrim (22), and Krishnan (14) etc. Besides these there are a number of archeological findings of ancient towns like Pataliputra (near Patna) and Harappa Mohenjo Daro (in Rajasthan) which now lie buried down in the Indo gangetic alluvium. Notorious and unstable behaviour of the Indus, the Ganges and the Brahmaputra system of rivers from time to time are well known. Sudden devastrous floods of great magnitude resulting sometime even in the declination of complete civilization (26) have also been reported.

Many attempts have been made recently (8,24,25) to analyse the factors responsible for peculiar behaviour of rivers and exceptional floods in the Indo gangetic plains. Flood frequency studies at certain headworks (2) and the past flood data have revealed that floods of exceptional discharge and magnitude occur only a few times in a century. Strangely enough the above catastrophe has not been explained from the point of view of the geotectonic movements.

The manner in which these geotectonic movements are developed and their ultimate affect on hydrography could not be studied because of the inaccessability of the basement rocks which are covered by a thick blanket of alluvium. However, based on the information available from the recent geological (19), geophysical (11), geodetic (10), aeromagnetic (1) and seismological records it has been possible to visualize the nature of the underlying tectonics. The views put forward here are the result of a study of all the above mentioned evidences in the light of the past seismic and tectonic history of the Indo gangetic plains. An aerial and land survey were also undertaken by the author to observe the behaviour of the rivers of the Ganga and the Brahmaputra system specially during high flood seasons.

STRUCTURE AND TECTONICS

Little is known about the exact nature of the geological history of Indo gangetic plains, because the whole expanse of these plains from the Punjab to Assam form only a thick deposit of Pleistocene and Sub Recent

*The term Indo gangetic plains also includes Brahmaputra Basin (9)

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alluvial deposits.

That the origin of these alluvial troughs has been intimately connected with the great orogenic movements of the late Tertiary times is a very widely accepted view. The idea of Eduard Suess (28) who considers it a "fore deep" i.e. a slight buckling down of the crust like a synclinorium in front of the newly arisen Himalayas has also received a general concurrence.

A glance at the generalized North - South and East - West cross sections of the trough as shown in Figs. 1 & 2 will give a rough idea of the overall structure and tectonics of the Gangetic trough. Fig. 1 represents only a diagrammatic North - South cross section of the Gangetic trough. On the north are the Himalayas proper and near the southern edge of it a series of successive boundary faults. To the right is the alluvial trough proper, the floor of which at first slopes downwards to the point of maximum depth (15,000 - 20,000 according to Oldham)(19) and then rises gradually upwards to the southern limit of the alluvium and the northern fringe of the peninsular table land. The maximum depth of the trough lies between L 82° to L 86°.

As shown in Fig. 2 which is a diagrammatic representation of a longitudinal cross section in E.S.E. - W.N.W. direction the trough lies hinged at two places. To the west are the Aravallis and their postulated N.E.N. extension below the alluvium. To the east lies the North - East extension of Garo - Rajmahal series of peninsular rocks. Strike directions of these two formations are shown in Map. 1.

As elucidated by the geodetic evidences, Oldham (19) pointed out that the position of the southern boundary of the trough at the eastern end terminate where Assam range impinges on the boundary of the Himalayas. The trough has a width of about 80 - 100 miles near Jalpaiguri and Ramchandpur. In a westward direction the southern boundary of the trough is fixed by the boundary of the rock and alluvium at Monghyr. Later it follows the general course of the boundary of the Peninsular rock area till L 80°. Near Agra and Delhi it runs almost N.N.W. and the boundary lies close to Hatharas. Westward of this line is merely a covering of a few hundred feet of alluvium over the rocky floor.

From the above description of the structure and general geomorphology it can be summarized that we have a fairly symmetrical trough which lies like an apron along the southern side of the Himalayas.

It may be pointed out that the trough whose form and dimensions have just been described is only a part of the great spread of alluvium known as the Indo-gangetic basin. The Indus basin, similar to the Gangetic basin though smaller in size runs along the foot of the hill ranges of the Baluchistan Arc. The two troughs are probably separated by a rock barrier, stretching under the plains of the Punjab to the Salt-Range as pointed out by the outcrops of old rocks in the Chimot and other hills which rise above the alluvium.

SEISMOLOGICAL FACTORS

Tectonic distribution

The subject of the origin of the great potential energy inside the rocks and further the manner in which it is translated into kinetic energy ultimately resulting in earthquakes, is much too conjectural and is beyond the scope of the present paper. The distribution of the active seismic zones in Indis along the southern flanks of the Himalayas and the northern border of the Indian peninsula which has also been folded, faulted and over-thrust during the Tertiary era indicate that most of the earthquakes originate because of the marked instability of the tectonic units. So much so, actual fractures of rock are known to have accompanied the tremors during the Cutch (India) earthquake 1819, (20) the Chaman (Baluchistan) earthquake of 1892(29) and the Assam (India) earthquake of 1897 (21).

Instability of the Basement Tectonics

The tectonic units of India and their inter-relationship with the seismicity of different areas have been discussed earlier (4,5,7,27). Confining mainly to the Indo-gangetic troughs, the present paper aims to focus the peculiar disposition of the various geotectonic features in the light of the rock behaviour phenomena in tectonic processes by which it would appear that the state of great stresses and potential fractures must be present to exist below the Indo gangetic alluvium.

As discussed in the preceding section Suess's idea of synclinorium which has later been confirmed by the geodetic evidences (10,11) explains most of the causes of tectonic instability. Viewing back the origin of these synclinoriums it has been stated (13,14) that they must have been formed by a slight buckling down of the crust in obedience to the pressure exerted on the borders of the peninsula by horizontal compressive forces.

As the rocks exposed at the surface are bent towards the south in the Himalayan Arc, towards the west and east in the Burmese and the Baluchistan arcs respectively the author is of the opinion that the direction of the horizontal compressive forces which were responsible for the downwarping movement, must have been in two directions i.e., East - West and North - South, perpendicular to each other. In such cases, when a mass is distorted into any new shape with respect to two compressive forces acting in two different directions, the dilation is only possible in one direction perpendicular to the two forces. Hence, the dilation must have been in a vertical position producing thereby - the anticlinoriums and synclinoriums. Incidently it may be mentioned here that the drop of the Garo Rajmahal Gap on the eastern side of the Gengetic trough and the Aravalli hills postulated extension on the west of it was probably the result of this vertical movement.

Obviously the elastic stresses developed at the time of this crustal deformation or distortion must be tensional at the outer arc and compressional at the inner arc of the synclinorium. As expected these stresses inside the thick elastic rock mass must have resulted in the shearing stress parallel to the surface which ultimately developed shear planes on

the flanks of the fold (Figure 3). This would naturally lead to the development of concentric shear planes with respect to the forces supposed to be still alive according to Oldham (19), Wadia (27,28), West (29) and other (5,13,14).

No doubt the process of elastico - ciscous flow assumes that on these concentric shear planes the shearing stresses rise beyond the limit set for flows and cause big ruptures. The amount of kinetic energy released then or in other words the severity of the earthquake shock depends upon the type of rocks and the thickness of the strata involved at the basement.

In the light of the explanation of instability described above the area of maximum instability must lie along the axis of the asymmetrical Gangetic syncline which runs in E.S.E. and W.N.W. direction very near the foot hill zone of the Siwaliks. This has actually been observed during the great Bihar Earthquake of 15th Jan. 1934.

> "The elliptical nature of the isoseismals strongly suggests the linear nature of the centrum or focal region and the occurance on one or more fractures below the Gangetic alluvium between Motihari and Purnea may safely be postulated" (p. 220) (4)

It appears that Hunter (12) described this area running exactly in the line of the present author's asymmetrical synclinal axis (E.S.E., W.N.W) as an area of underload. He further stated (loc. cit.) that the stress differences are caused by the loading anamoly between the zone of underload and overload. He has not actually shown the direction of stresses but presumably the two areas shown by the two points in Fig. 4 drawn herein indicate that they must act into two different directions, creating zone of stress in between the two. If this is correct, the epicentre should not lie exactly below the area either of underload or of overload (loc.cit.) but in between the two.

The above view point is indirectly supported by the observations of E. A. Glennie (10,11) that the anamelies are not due to the low densities of the alluvium but due to a deep seated phenomena whereby the granite, basalt, and dunite layers of the earth crust have been depressed over this area and elevated further south.

Another noteworthy tectonic feature which deserves mention in this section is the existence of the Aravalli ranges extending across the region where the Gangetic trough was subsequently brought into being. This would certainly produce a pronounced effect on the instability of structures by introducing a variation in the strength of the earths crust and consequently a difference in the resistance which it would offer to the forces by which the trough was produced and also to subsequent subsidence. Similarly the interruptions on the eastern side of the trough can be visualized by the North - East extension of the peninsular rocks.

SECULAR CHANGES IN LEVEL

From Fig. 4 it will be noticed that the process of splitting up of the original layer into more and more independent arcs will allow continuous

bending of the strata without further increasing the stress inside the rocks so that the elastic state of the rigid rocks is never lost.

In support of the continuous existence of stresses to cause this further bending of the trough we find a general concurrence among geologists and seismologists that the Central Himalaya & Outer Himalayas and the Gangetic basins are under great stresses, which are maximum between the southern boundary of the Extra-Peninsula and the northern fringe of the Gangetic trough:

> "At any rate, it is clear that the northern rim of the great trough is under considerable tectonic strain due to progressive downwarping with the greatest subsidence where it merges into the foot of the mountains" (28)

The stresses remain in the form of potential energy inside the elastic strain of rocks.

Huge quantities of sediments are brought down by the Himalayan rivers and desposited in their respective sedimentary basins. This process of erosion and sedimentation does creat an additional stress over the basement rocks. These additional stresses go on accumulating as the process of sedimentation and erosion proceed for a number of years till they in conjunction with the original potential stresses overcome the elastic strain limit of the rocks when further splitting and rupture of the substratum takes place. It is mentioned (29) that sedimentation load does not play a significant role in stress formation. But the geological processes of erosion and sedimentation which have ultimately resulted in the deposition of thousands of feet of alluvium are not so insignificant and do creat additional stresses which act like a trigger of a gun to release the stresses - to cause rupture - earthquake and ultimately subsidence.

Although it is difficult to estimate quantitatively the actual amount of differential movement of the upper crust in obedience to the downwarping at the base, it is quite certain that such a great thickness of alluvium must have been deposited by a process of secular subsidence and alluviation.

Such secular changes in level were actually observed during the earthquake effects of North Bihar in 1934 and a number of illuminating examples of subsidence have been reported (7).

The area of the belt of subsidence was reported (loc.cit.) to be 4,700 sq. miles in Bihar and extended for about 190 miles in a W.N.W. and E.S.E. direction which again is exactly the same as the direction of synclinal axis.

Examples of such secular subsidences are not rare in other countries also. New Madrid earthquake has afforded indisputable evidence of vertical subsidence and the area of sunk land are still preserved along the alluvial depressions west of the Mississippi Valley over a total extent of about 1000 sq. miles. The depression was reported to be 20 feet. (7)

The Cutch earthquake of 1819 (20) resulted in the elevation of an

area 50 miles long and 20 miles wide and 10-26 feet in height. Concurrently with this elevation, the area to the south subsided.

HYDROGRAPHIC INFLUENCES

Rivers alter their courses and exhibit peculiar behaviour at the slightest provocation. In the preceding description we have seen that the geotectonic movements which result in secular differential changes in ground level must have a pronounced effect on the hydrography of the Indo-gangetic plains. Ground water-table was raised and rivers changed their courses during the North Bihar earthquake of 1934.

River Courses

In obedience to these geotectonic movements rivers in Indo-gangetic plains have been changing their courses since late Tertiary times. These changes in fact have resulted even in complete reversal of the direction of flow of the chief rivers in the Indo-gangetic plains. The river named "Siwalik River" by Pilgrim (23) and the "Indo-brahm" by Pascoe (20) was a great north-west flowing river lying at the southern foot hills of Himalayas from Assam to the farthest north-west corner of the Punjab and then flowing towards the south met the gradually receding Miocene sea of Sind and Punjab.

Consequent to the Post-Siwalik movements a dismemberment of this river was brought about and ultimately three subsidiary systems - (1) the Indus (2) the five tributaries of the Indus and (3) the Ganges were formed. The Indus has been changing its course from time to time.

> "The main channel of the Indus upto 1800 A.D. flowed through the middle of the Thar. In that year it split up into two channels, and of these (the Khedewari) which was the main waterway upto 1819 was blocked by an earthquake. By 1837 the Kakaiwari was the main stream but this also became blocked in 1867. In 1900 the chief channel was the Hajamro".(14)

There are both physical and historical evidences to show that the sacred Saraswati river of Vedic times - now almost a lost river - was once a mighty river and flowed through the eastern Punjab and the Rajasthan. An old bed some 600 miles in length is still traceable from Ambala through Bikaner and Bahawalpur to Sind. (17)

The Yamuna used to discharge into the Indus system, but due to the geotectonic movements in Sub-Recent times it took more and more easterly course and ultimately joined the Ganges at Allahabad (27).

The 'Sutlej' which was once an independent river never joined the river Indus but joined the Beas near the south-west corner of Kapurthala in 13th century. The Beas also had a different course which can still be traced.(14)

Patliputra is said to have been situated at the junction of five rivers- the Ganges, Gogra, Gandak, Son and the Pumpun. The city now lies buried beneath the alluvium near the modern Patna and the junction of these rivers with the Ganges are widely separate.

The Tista which was once a tributary of the Ganges, became a tributary of the Brahmaputra after the disastrous floods of 1787. The Brahmaputra which used to flow east of Madhopur jungle now joins the Padma much further west (14).

Similarly there are many other instances of great Recent and Sub-Recent changes in the course of rivers in the Indo-Gangetic plains. The Kosi in particular has shifted its course from time to time and has swept an area of nearly 4,000 sq. miles.

Regarding the hydrography of these plains to-day we find that it is directly dependent upon the various hydrostatic gradients which appear to follow the structural pattern of the floor of the troughs. As has already been discussed the Gangetic trough is hinged towards, north by the Himalayas, towards south by the Peninsular rocks. towards west by the Aravallis and towards east by the Garo-Rajmahal extension. Consequently four gradients lying approximately in S.W., N.E., S.E., and N.W. direction, and having the resultant gradient in E.S.E. - W.N.W. direction which is the general drainage of the Ganges, have been established. The resultant gradient of the Indus basin is towards south west.

Floods

A query as to why we have had more and greater floods in the Gangetic basin during recent times has been a much discussed problem among hydrologists and engineers. Technical Sub-Committees (24,25) and high level committees (8) appointed from time to time could not trace satisfactorily the actual cause of sudden excessive flood cycles. It has been observed during the recent cycle of floods that the rate of rise of water has been very rapid, floods subsiding slowly and leaving water logged areas. New areas have been invaded by the inundating flood waters and the abandened channels got new life. In addition it is reported that the floods mostly occur during the later part of the rainy season.

From the earlier discussion it is clear that it is very difficult to estimate quantitatively the actual amount of secular differential changes in ground level. But it is certain that these geotectonic movements do establish from time to time a new and significant hydraulic gradient equilibrium sufficient to cause disturbances in the original flowing and transporting capacity of the rivers of these basins. The maximum amount of subsidence in the Gangetic basin can be expected to be between L 84° to L 86° in E.S.E. and N.W.N. direction. Naturally the first and the maximum gradient in south west direction (Map 1) gets considerably increased with the differential rise of the hilly tracts lying towards the north. On the other hand the resultant slope in E.S.E. and N.W.N. direction after about L 84° gets relatively reduced. In support of this assumption we find that the western part of the Gangetic basin is about 500 to 900 ft. above the sea level, while the level of the eastern part at Allahabad where the Yamuna meets the Ganges is only 340 ft. above the sea level. Allahabad is about 512 miles from Clacutta and therefore the slope works out to less than one foot per mile. In the light of these geotectonic movements the slope of one foot per mile gets further reduced which is difficult to measure by ordinary survey instruments.

As a result of these changes in various gradients rivers drain off their catchment area with a changed degree of efficiency. Rain water from hilly tracts is brought down with a higher velocity than before. This water further meets a reduced gradient in S.E.S. direction which ultimately results in sudden rise of the water level and all other phenomena connected with the recent floods.

The decreased slope and blocking up of the drainage helps in more underground percolation of water, thereby recharging of the subsurface reservoir becomes more efficient, till it gets almost saturated in the early part of the rainy season. This in conjunction with the slight rise of water table caused by the tectonic movements explains the recurrence of floods in the later part of the rainy season.

DISCUSSION

Silt deposition, deforestation, excessive rain fall, land slides or other causes are sometimes thought to be the main cause of the recent havoc of floods and the changes in the river courses.

Silt deposition is one of the factors which can decrease the slope. But the sudden excessive silt deposition can only be an effect to some other basic cause. The slow and continuous phenomena of silt deposition does not become effective so suddenly in all the rivers at the same time. Further any amount of silt deposition will not influence the slope as long as the river has got a movable bed. This is in conformity with the idea of Leet.

> "If we go beyond casual observation and gather accurate data on width, depth and velocity and discharge of a stream from its head waters to its mouth for a particular stage of flow-say flood or low water - we find again that the change follows a definite pattern and that the depth and width increase down stream as the discharge increases."(15)

Suppose a silt particle is deposited at point A as in figure 5. This particle will tend to reduce the velocity of water from A to B according to the laws of sedimentation. Since deposition is directly related to the velocity same size fractions should be deposited gradually up-stream upto B and then upto C,D,E etc. In short the stream tries to maintain the original gradient in order to maintain the flow.

Deforestation, excessive rainfall, land slides, etc. can only be of local importance in one or two rivers but could not be on regional basis as has been happening in case of the Indo gangetic plains.

CONCLUSIONS

The following conclusions can therefore be drawn from what has been set out in the preceding sections.

1. The Geotectonic movements which were originally responsible for the formation of the Indo-gangetic plains as well as for the present state of great instability are due to the two horizontal compressive forces acting perpendicular to each other in north-south and east-west directions.

2. The two horizontal compressive forces have resulted in the formation of zones of concentric shear planes along the axis of the asymmetrical syncline of the Gangetic trough.

3. Earthquakes originate due to the rupture of the strata created by the tensional forces operating at the outer arc of the Gangetic synclinorium. The tensional forces are primarily due to the two horizontal compressive forces and secondarily to the process of erosion and sedimentation.

4. As a result of the geotectonic movements in the Indo-gangetic plains great amount of secular changes in ground level have been continuously taking place.

5. Hydrography of the Indo-gangetic plains has been profoundly effected since post-Siwalik times as a consequence of these geotectonic movements. Changes in the main drainage lines and recurrence of the flood cycles are attributed to the secular differential changes in the hydrostatic gradients of the rivers of Indo-gangetic trough.

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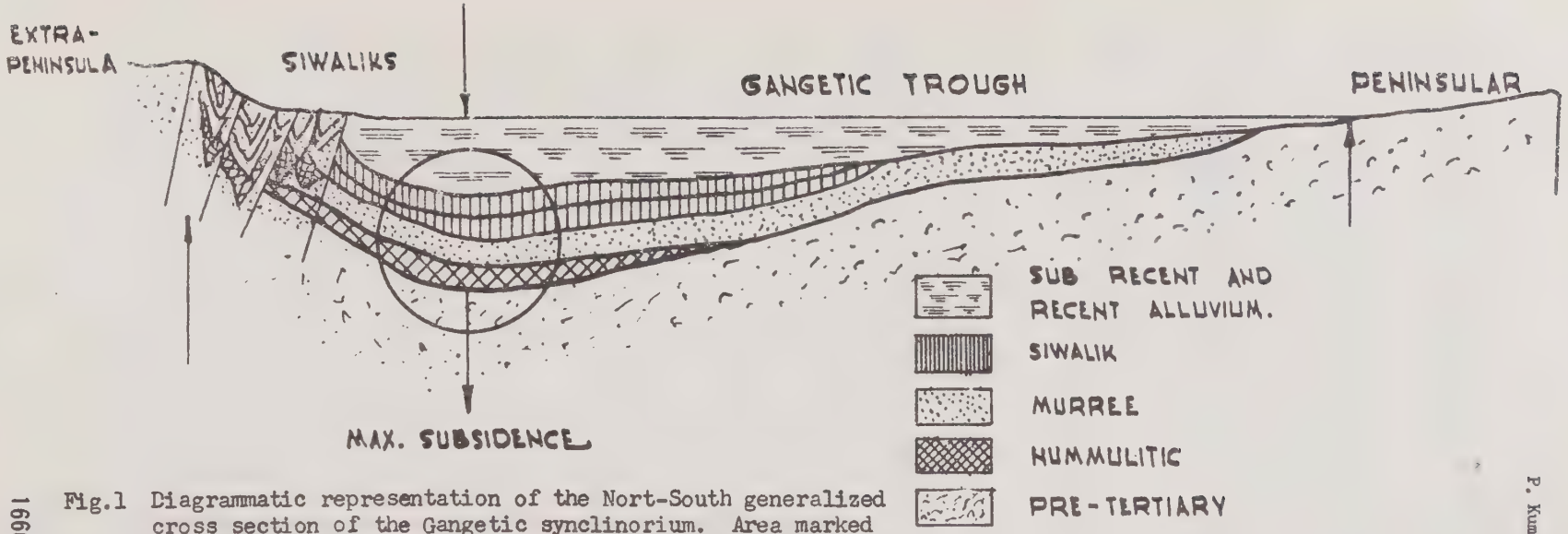


Fig.1 Diagrammatic representation of the Nort-South generalized cross section of the Gangetic synclinorium. Area marked by the circle represents the zone of great instability.

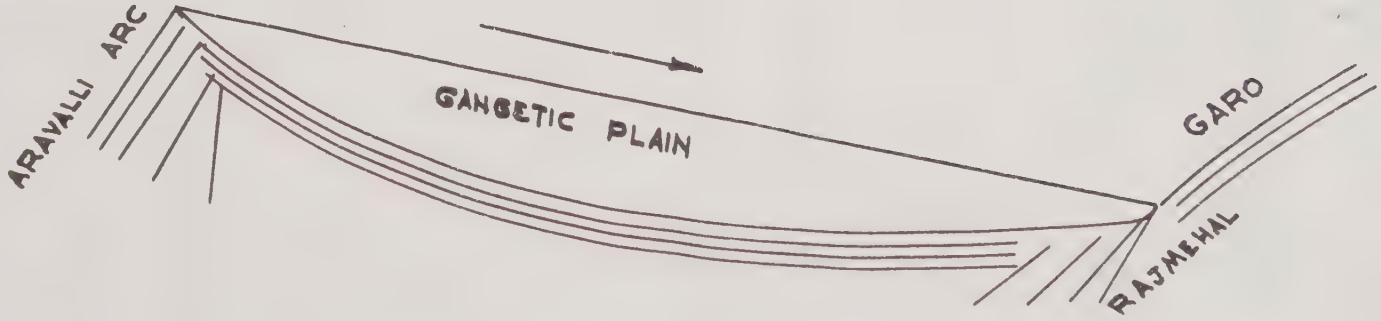


Fig.2 Diagrammatic representation of the E.S.E., W.N.W. generalized cross section of the Gangetic synclinorium.

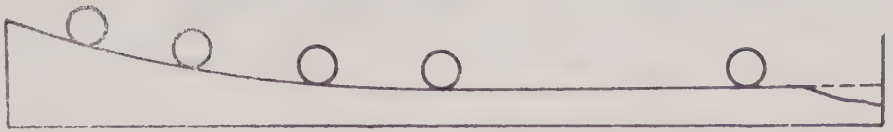


Fig.5 Sedimentation in an open stream.

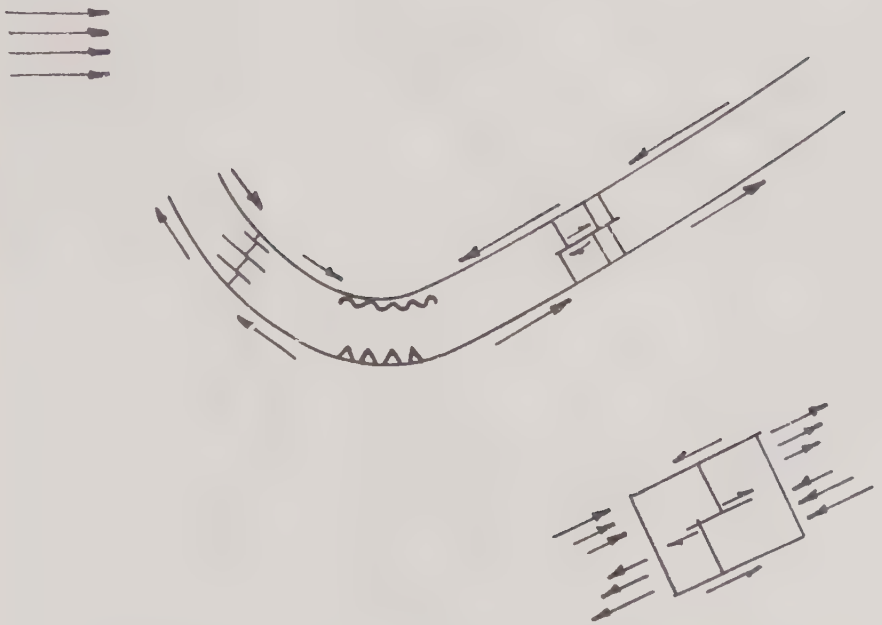


Fig.3 Elastic stresses in Gangetic syncline leading to the development of concentric shear planes.

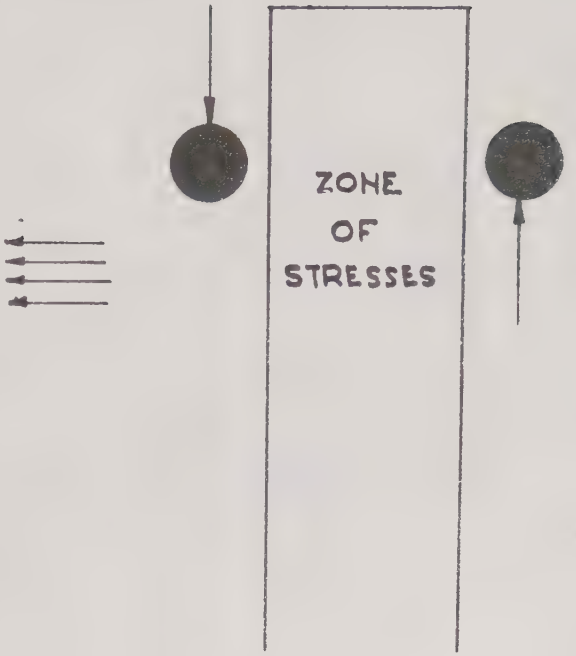
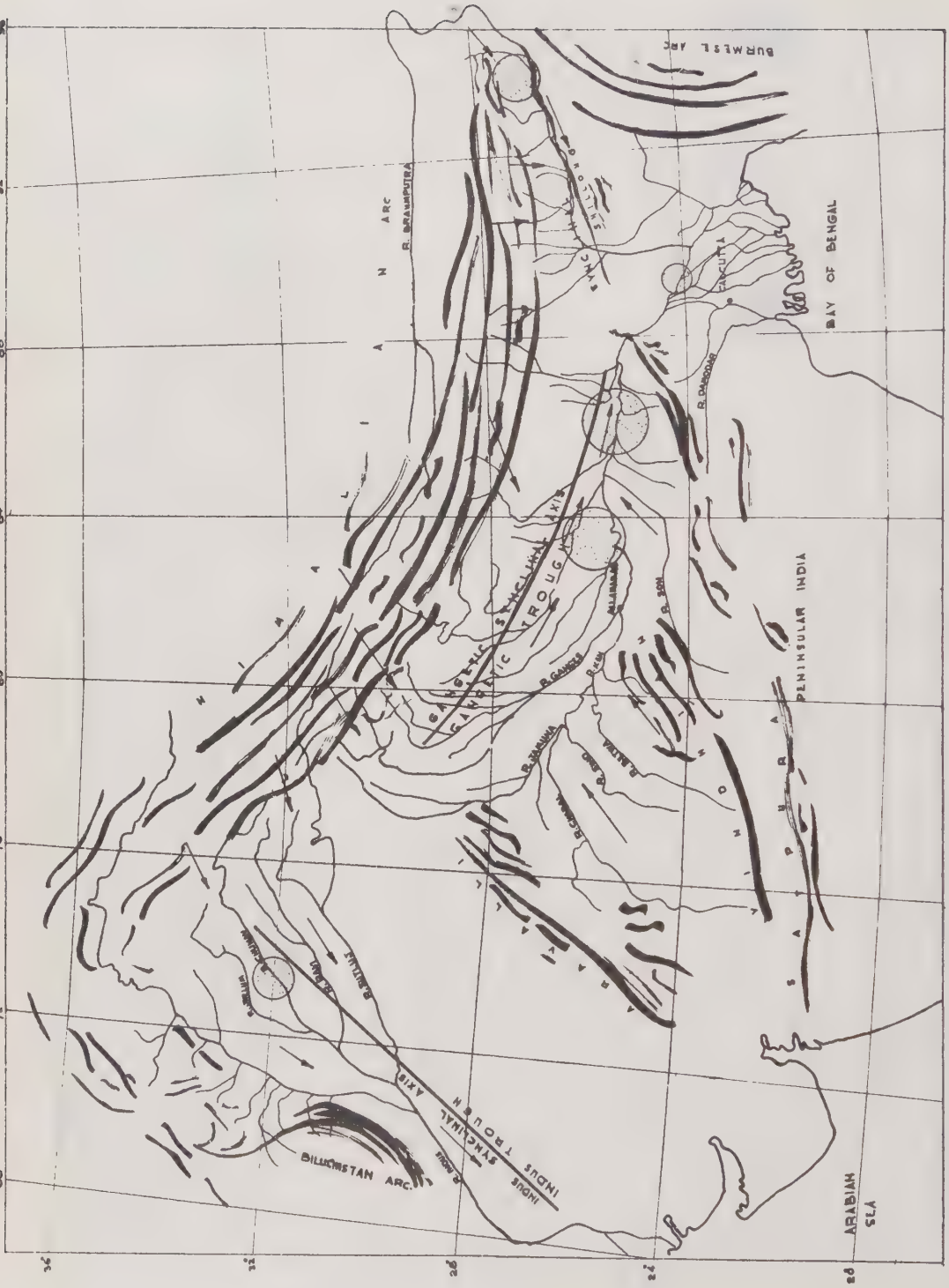


Fig. 4 Directive forces and the zones of stress presumed according to the idea of under-load and overload.



Map:1 Showing the hydrography of Indo gangetic plains. Arrows mark the various hydro-static gradients. Area lying in circle indicate the total flood damage during 1950-56.

EARTHQUAKE INSURANCE IN THE UNITED STATES - A REAPPRAISAL

By

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In considering earthquake insurance in the United States, it should be recognized that the insurance underwriter, the seismologist, and the engineer all have similar objectives - that of understanding more about earthquakes.

Although much has been learned by the seismologist in the last 50 years, only part of this knowledge is of use to the insurance underwriter and for this reason earthquake insurance rating as a science is much in need of further study and comprehension.

The problem of most insurance underwriters is that they do not possess sufficient knowledge to understand many of the technical aspects of seismology. On the other hand, the seismologist may not readily appreciate some of the problems of the underwriter, and this paper is directed at the objective of informing seismologists of some of our problems with the hope that we in turn will benefit from their analysis and advice.

FACTORS TO BE CONSIDERED IN ESTABLISHING EARTHQUAKE RATES

In general there are six factors which are considered in making fire and other property insurance rates:

1. Construction
2. Protection - both public and private
3. Occupancy
4. Exposure
5. Element of time - frequency of loss occurrence
6. Credibility of available statistics

The factors which must be considered in establishing earthquake rates are directly related to these factors, although they are often described in different terms. Each factor will be discussed below both in terms of its general significance in property insurance rate making and its special importance in determining earthquake insurance rates.

Construction

Construction has long been recognized as of major importance in determining earthquake rates and is sometimes referred to as the damageability factor or the relative resistance of various types of buildings to earthquake shock. Most earthquake rating schedules recognize 8 classes of construction ranging from the structural steel frame building which has been designed to resist earthquake shock to the poorer type of building constructed of tile, concrete block or adobe bearing walls. Bridges, reservoirs, dams, and brick stacks, as well as glass greenhouses and wood tanks on wood towers, also fall into the higher rated classification.

Other important factors which have been recognized include the natural period of vibration of the structure, its height, the type of ground upon which the building has been built, the presence of unanchored parapets or overhanging cornices, and the condition and type of cement mortar used in buildings of masonry construction.

The work of the late John R. Freeman in this important field is well known to most of you. Some of the important conclusions reached by Mr. Freeman are as valid today as they were in the late 1920's and early 1930's and are repeated here because of there significance to the overall problem of proper insurance underwriting.

1. A well designed and well built building of reinforced concrete having a rigidly braced steel frame and which is less than 100 feet in height, will resist an earthquake shock as violent as any that have occurred in the United States or Canada with only minor resulting damage.

2. Well designed and well built wood frame dwellings on good foundations will sustain only minimum damage in a major earthquake.

3. Structures of cement block although one or two stories in height are likely to be badly shaken and walls of adobe block have proved extremely weak in resisting earthquake shock. (It is significant that the recent major earthquake in Agadir has again proven the validity of this statement).

4. Ordinary brick veneer on wood framing will sustain major damage by earthquakes of even relatively moderate intensity, and hollow tile walls and partitions and ordinary brick chimneys are among the first parts of a building to be damaged.

5. Buildings on good concrete foundations invariably show less damage than those constructed upon piers or light footings.

Protection or the Resistance Factor

Although in the fire and allied lines field we generally think of protection as related to the presence or absence of automatic sprinklers and the degree of public protection available, protection is also recognized in earthquake insurance although perhaps it is more often thought of as the resistance factor or the fact that through the inclusion of special features buildings can be constructed which will safely resist earthquake shock.

Well informed managements always consider the need for automatic sprinkler protection when constructing new buildings because they are aware of the differential in fire insurance rates between a sprinklered and unsprinklered building and because sprinkler protection is of significant importance in reducing loss of life as a result of fires. Equally well informed managements building new

plants in earthquake areas should also recognize the need for earthquake resistant construction. Fortunately building codes on the west coast of the United States make such recognition mandatory in most areas. Unfortunately such requirements are far from universal and needless loss of life and damage will continue at places such as Agadir until better regulations are adopted.

Occupancy

In the United States separate building and contents base rates for fire insurance are developed for more than 100 different statistical classifications. Earthquake rating schedules are not as detailed but the kind of occupancy is important in determining earthquake contents rates. The earthquake insurance rules of the Pacific Fire Rating Bureau provide five classes of contents and the rates for classes most susceptible to earthquake damage are from 5 to 12 1/2 times as great as the rates for contents which are least susceptible depending upon the type of building construction involved. The Factory Mutual Fire Insurance Companies, who are insurers of large industrial risks, use their own earthquake rating schedule which has three classifications for contents. Contents rates provided by this schedule bear a constant relationship to the building rate for earthquake insurance and are 100%, 75% or 50% of the building rate depending upon the susceptibility of the contents to earthquake damage.

Present knowledge does not indicate the need for any further refinement of earthquake rating schedules insofar as occupancy is concerned. The major problem is related to exposure and frequency of loss occurrence and these subjects will now be discussed.

Exposure or Area Factor

Exposure is also recognized in establishing earthquake rates although it may be more generally thought of as the area factor or the territory within which damage occurs as a result of a single earthquake. From the standpoint of the fire risk, exposure is considered from three viewpoints, namely internal exposure from other risks in the same building, external exposure such as adjoining or nearby buildings, and conflagration exposure. The conflagration exposure from fire has been considerably reduced since the 1906 San Francisco disaster. Improvements in building design and fire fighting techniques have in fact, so reduced this exposure that it is very unlikely that we could have a conflagration today which would, for example, approach the damage sustained in the great Chicago fire of 1871.

Surveys are made of all large cities in the United States today and existing conflagration areas are mapped out and are used by underwriters in considering this exposure and most insurance companies limit their participation on risks in such areas.

In earthquake insurance we are not as fortunate in having such information and we need to know the areas over which violence may be sufficient to destroy structures or damage them in any single earthquake.

Studies made by John R. Freeman indicate that the area of destructive violence in any one earthquake has always been relatively small and it is highly probable that this area will never exceed 2500 square miles when considering earthquakes in the United States. Seldom have earthquakes caused important damage to structures at distances of more than 25-50 miles from the center of the distrubance and major damage is generally restricted to a narrow belt not more than 10 miles wide.

In the San Francisco earthquake of 1906 this belt was about 200 miles long, but the area of greatest intensity was still of modest size. The report of the California Earthquake Investigation Commission in connection with the 1906 San Francisco earthquake, indicated the following areas of intensity, (Rossi-Forel scale):

Zone	X	- 400 square miles
Zone	IX	- 3000 square miles
Zone	VIII	- 9000 square miles

Another severe earthquake, the Santa Barbara earthquake of June 29, 1925, involved a much smaller belt approximately 1/4 mile wide and two miles long representing the area of severest intensity within which the major damage occurred.

The New Madrid earthquake of December 16, 1811, was also restricted to a fairly narrow belt perhaps 100 miles along the Mississippi River. The local severity of the New Madrid earthquake was intensified, however, by the peculiar circumstances of the geological and sedimentary conditions of the area involved.

The Assam, India, earthquake of 1897 was felt over an area of about 1 3/4 million square miles and yet reports indicate that the epi-central area of this earthquake was only about 200 miles long from east to west and about 50 miles in width. One report indicated that the maximum intensity of X Rossi-Forel scale extended for a distance averaging only about 70 miles from the epi-center.

It isunfortunate that the total area over which a major earthquake is felt is often misinterpreted as being directly related to the amount of damage which will result. It must be recognized that comparisons of relative damage of various earthquakes must take into account the type of construction involved and the type of ground upon which the structures have been built. These factors should not be confused with the area of major destruction or the area within which there is the possibility of major destruction.

Element of Time - Frequency of Loss Occurrence

In the property insurance field and particularly in fire insurance, it is recognized that loss trends do exist, but because of the usual frequency of fires and the large amount of insurance written, the extremes are not as great in fire insurance experience as they are in earthquake experience. Reference to the attached chart showing a comparison of the fire and earthquake loss ratios

for the period 1916-1957 shows that the fire loss ratio for the United States is generally contained within a range of ratios of from 40% to 60%. The absence of major conflagrations in the last 40 years has contributed to this result.

As would be expected, the earthquake loss ratio for the State of California for the same period shows a much wider range of fluctuation, having several minimums of 0 and a maximum of 118% representing the loss ratio for the year 1933 as a result of the Long Beach, California, earthquake.

Loss Ratios for extended coverage insurance (includes the perils of windstorm, explosion, riot, etc.) are more susceptible to fluctuation than fire insurance and although not shown on the chart, in the period 1940-1957, extended coverage insurance for the United States had a loss ratio range of from 21.8% to 63.3%. Extended coverage experience on a State basis, would produce greater fluctuations and has some of the problems of earthquake insurance inherent in its makeup although a much wider premium base exists.

A company writing dwelling insurance can foresee with reasonable accuracy the number of dwellings that will be subject to damage by fire and windstorm and can estimate within reasonable bounds what the loss ratio will be over a five or ten year period. We have already pointed out that the problem of conflagration is not serious today in most areas, and as long as the underwriter obtains a reasonable spread of risk, he need not be too concerned with this factor.

In extended coverage insurance a single windstorm may involve a wide area and losses of substantial proportion have occurred in recent years in various parts of the United States. Most companies, however, limit their insurance writings in areas susceptible to hurricanes and the spread of risk evens out the experience. Extended coverage insurance is widely purchased throughout the United States today, thus providing a firm base for statistical analysis. It is significant to note, however, that the November 25, 1950, windstorm in the northeastern United States caused damage resulting in claims in excess of 200 million dollars, whereas the San Francisco earthquake of 1906 resulted in earthquake damage estimated at 90 million dollars when converted to the 1950 equivalent.

In earthquake insurance the frequency or infrequency of earthquakes in certain areas provides a serious problem for the underwriter. While he can see that reasonable underwriting based on well established principles will produce reasonable loss ratios for fire and extended coverage insurance over say the next five or ten years, he cannot know what the earthquake experience of his company will be over the next five years or even the next fifty years. Since most underwriters don't have a working span of fifty years,the problem is discouraging both to the individual underwriter and the company he represents. The Company also knows that any excess profits from a period of good earthquake experience will be eaten up by taxes, because no provision has been made for accumulating special earthquake reserves.

The problem of frequency, particularly of earthquakes of major intensity, is a matter of most concern to the underwriter and yet is probably the factor most difficult to predict at the present time. In the preparation of this paper the author spent considerable time compiling a list of earthquakes of various intensities for different regions of the United States. Reference was made to "Earthquake History of the United States - Part I Continental United States Exclusive of California and Western Nevada" - revised (1956) edition and "Earthquake History of the United States - Part II - Stronger Earthquakes of California and Western Nevada" revised (1951) edition. Reference was also made to the annual publication "United States Earthquakes" for the years 1951 through 1956 to provide an up-to-date record of the earthquakes in California and western Nevada.

Earthquake History of the United States - Part I provides information based on the Modified Mercalli scale whereas Earthquake History of the United States - Part II provides information based on the Rossi-Forel scale. All California earthquakes were converted to the Modified Mercalli scale in order to provide a comparable base. Table I shows the number and frequency of earthquakes of various intensities for the several regions of the United States.

The table indicates, for example, that earthquakes of intensity ten and over (Modified Mercalli scale) have a frequency of 1 in 13.3 years in California as compared to a frequency of 1 in 293 years in the eastern part of the United States with the other regions falling between these two extremes.

The chart also shows that in California for earthquakes of intensity 9 and over (Modified Mercalli scale) the frequency is 1 earthquake in 7.5 years; for intensity 8 and over, the frequency is 1 in 2.9 years; and for intensity 7 and over, the frequency is 1 every 1.35 years.

If this information is plotted using intensity as one scale and frequency as the other, it is possible to compare the areas represented by the curves for the different regions involved. The resulting relationships using California as unity are as follows:

California	- 1.00
Western Mountain Region	- 0.620
Northeast	- 0.097
Central	- 0.082
Washington-Oregon	- 0.056
Eastern	- 0.034

If these relationships are in any way valid insofar as the indication of the relative frequency and intensity of earthquakes in various areas of the United States, then it would appear, for example, that earthquake rates in the Northeastern United States should be about 1/10 of the rates charged in California. It is recognized that the above relationships are based on giving equal weight to earthquakes of various intensities which in reality, is probably not a fair assumption. Therefore, it was

decided to take the available information and compare the average damage in dollars for earthquakes of various intensities. Table II shows the results of these computations. If these figures could be accepted as valid, they would indicate that earthquakes of intensity 10 (MM) and above, would cause earthquake damage which was 26.5 times the damage caused by earthquakes of intensity 7 (MM).

It is recognized that the credibility of information available does not warrant any final conclusions. However, such study as the author has been able to make does indicate that the whole question of frequency and the relative damage caused by earthquakes of various intensities should be further investigated as a means of aiding the insurance underwriter and the insurance buyer. The seismologist can be of tremendous help in solving these problems.

Credibility of Statistics

The problem of the credibility of rate making statistics is well known to most actuaries and underwriters. Earthquake insurance statistics pose a particularly difficult problem and it has not received as much study and analysis as other types of property insurance.

In the field of fire insurance rate making certain credibility guides have been adopted and three of these will be mentioned to show the diversions of opinion and to point out some of the problems involved. The New York Insurance Department, for example, uses a credibility table as follows:

5 Year Premium	5 Year Credibility - %
$6,000,000 and over	100%
4,860,000 - 5,999,999	90
3,840,000 - 4,859,999	80
2,940,000 - 3,839,999	70
2,160,000 - 2,939,999	60
1,500,000 - 2,159,999	50
960,000 - 1,499,999	40
540,000 - 959,999	30
240,000 - 539,999	20
60,000 - 239,999	10
0 - 59,999	5

The Texas State Board of Insurance has adopted a different credibility table for fire insurance as follows:

Premium Volume (5 years adjusted earned)	Per Cent of Credibility
$1,000,000 and over	100%
900,000 to $ 999,999	95
800,000 to 899,999	90
700,000 to 799,999	85
600,000 to 699,999	80
500,000 to 599,999	75
450,000 to 499,999	70
400,000 to 449,999	65

Premium Volume (cont'd)	Per Cent of Credibility
$ 350,000 to $ 399,999	60%
300,000 to 349,999	55
250,000 to 299,999	50
200,000 to 249,999	45
150,000 to 199,999	40
100,000 to 149,999	35

The rules for the use of the Texas Credibility Table also provide that when a class or sub-class is not fully credible on its volume alone, then in addition and where applicable, the following may be considered:

a. A longer than 5-year record
b. Trends as disclosed in the record
c. The loss potential, i.e., whether the units of experience are such that one loss may disrupt the whole record. This involves study of the nature of risks presented and units of volume insured.

The Factory Mutual Companies have given considerable study to this same problem, and for the specialized highly protected risk, which constitutes a major portion of their business, they have adopted a credibility table based on average amount of insurance in force. Using a ten year period, 100% credibility is reached when the average amount of insurance in force for any particular class reaches 500 million dollars. Converted to an equivalent earned premium volume, this would represent 100% credibility at approximately 2 1/2 million dollars earned premium for a ten year period or $1,250,000 earned premium for a 5 year period.

It is thus obvious that even in the fire insurance field no fixed universal standards have been agreed upon regarding the minimum amount of premium needed for 100% credibility of statistics.

It is significant, however, that the 5 year earthquake premium writings in California are in excess of 26 million dollars, and all of the above credibility tables would indicate that full credibility would be reached when 5 year premium volumes reach 6 million and over.

Because of the relative infrequency of severe earthquakes, it would be unfair to compare the credibility problem of earthquake statistics to fire insurance statistics. However, the marked increase in extended coverage insurance premium writings in recent years have given the insurance companies statistical experience in a field which has many of the same problems as earthquake insurance. Whereas fire insurance premium writings have tripled in the period 1940-1957, extended coverage writings have increased 12 1/2 times, giving the companies a much broader base for statistical analysis.

The fact that extended coverage losses in excess of 200 million dollars have been paid in connection with one catastrophic

storm as compared to an estimated 90 million dollar loss for the 1906 San Francisco earthquake in terms of 1950 dollars, should provide an impetus for a reappraisal of the earthquake rating problem.

OTHER FACTORS TO BE CONSIDERED IN EARTHQUAKE INSURANCE

Certain other factors influence earthquake insurance to a greater degree than other fields of property insurance. Two of these are worthy of discussion, namely the attitude of the insurance buyer and the catastrophic nature of earthquake insurance.

Attitude of Insurance Buyer

If the reader will refer to the chart showing a comparison of fire and earthquake written premiums in the United States for the period 1916 to 1957, it will be recognized that certain trends exist. It is to be noted that fire insurance written premiums in the United States generally follow the economy and that in the period 1916-1929 a peak was reached followed by a sharp decline as a result of the 1929 stock market crash and the following depression years. Fire Insurance premiums leveled off in the early thirties and then started a gradual climb in the early forties, followed by a sharp increase as a result of the war years, reaching an all time peak in 1951.

During this period there has been a gradual downward trend in average fire insurance rates in the United States, and this is one reason for the apparent downward trend of premium writings for the period 1951 to date. .In actuality, the amount of insurance at risk or the value of property insured has undoubtedly increased during this period, although premium writings have been reduced because of decreases in rates. Some of these rate decreases are the result of experimentation in the multiple peril field by both fire and casualty companies.

We also have a record of earthquake insurance written in the State of California for the same period of time. The figures for earthquake insurance written on a Nationwide basis are available for a shorter period of time, but unfortunately there are gaps in the published figures for the 1940's. However, California still represents more than 90% of earthquake written premiums in the United States and provides a good comparison of trends.

It is to be noted that the earthquake writings in California in 1916 were non-existent and the writings in the period up to 1924 were practically negligible. As a result of the Santa Barbara, California, earthquake in 1925, the premiums jumped to a substantial figure. This was followed by another downward trend which was reversed slightly as a result of the Long Beach earthquake of 1933 and the Imperial Valley earthquake of 1940.

It is interesting to note that the general shape of the California earthquake written premium curve follows closely the shape of the fire insurance premium curve for the United States indicating that the general economy still has an affect upon the writing of this type of insurance. The curve also shows without question that serious earthquakes have an affect upon the insurance buyers attitude with respect to earthquake insurance,

and that generally speaking, earthquake premium writings increased significantly after every serious earthquake. These increases have been less prominent in later years but still exist.

The general picture insofar as earthquake premiums are concerned is one which indicates that insurance buyers are not convinced of the need for earthquake insurance and are prodded into buying coverage only as a result of serious earthquakes or the requirements of mortgagees.

This same attitude can be further emphasized by studying the relationship of fire insurance premiums and earthquake premiums in California. The total fire insurance premiums written in California in 1956 amounted to approximately 115 million dollars and if we estimate an average rate of 35¢-40¢ per $100 of insurance this indicates that the value of property insured is approximately 40 billion dollars. The 1956 earthquake written premiums in California were slightly in excess of 6 million dollars, and estimating an average earthquake rate of $1.00 at 50% insurance to value, would indicate the value of property insurance against earthquake is 1.2 billion dollars or less. The conclusion, then is that less than 5% of the property in California insured against by fire is also insured against the peril of earthquake. On a nationwide basis the figure would be much less.

The Companies thus find themselves in a quandry. The present level of earthquake premium writings is not broad enough to justify significant rate decreases. On the other hand, if earthquake rates were reduced and earthquake insurance was written on a broad scale, the companies would be better off and the public better protected. Unfortunately, there is no guarantee that a reduction in earthquake rates would result in large scale buying of earthquake insurance.

Another peril which offers many of the same problems is that of flood. The Factory Mutual Fire Insurance Companies, as a result of serious floods in certain areas throughout the United States, have on several occasions made detailed surveys of the requirements of their insureds with respect to flood insurance. The response to these inquiries indicated in practically every case that the demand for flood insurance was almost non-existent. Those who did wish to purchase such coverage, if it was made available, had the greatest exposure to loss and even these prospective buyers would be willing to pay very little for the coverage afforded.

Earthquake Insurance - A Catastrophe Coverage

The attitude of the insurance buyer as discussed in the preceding paragraphs bears out the well established conclusion that earthquake insurance is a catastrophe type of coverage. As a catastrophe coverage, it is generally recognized that deductibles which require the buyer to assume a certain part of the loss are a necessity if insurance rates are to be within reach of the average buyer.

A mandatory deductible on buildings is needed to eliminate

arguments over the cause of minor cracking of plaster and other maintenance deficiencies which are not generally caused by earthquakes but are the result of building age, and are often caused by a slight settlement of the building itself.

The stock company schedules used for earthquake insurance on the Pacific Coast of the United States require a minimum deductible of 5% with higher deductibles for the poorer types of construction. It is significant to note that the Factory Mutual Companies permit lower deductibles under their rating schedule with a minimum of 1% on the best type of building. The Factory Mutual Companies are also willing to insure contents against the hazard of earthquake damage on a no-deductible basis if the buyer so desires.

TREND IN EARTHQUAKE RATES IN THE UNITED STATES

In 1916-1917 the Board of Fire Underwriters of the Pacific adopted an earthquake rating schedule for a straight earthquake policy (that is, a policy covering the peril of earthquake only as opposed to the more generally accepted method today of endorsing a fire policy to include the peril of earthquake). This schedule contained three building classifications with annual rates as follows:

Class A and B steel frame buildings	- 25¢
Class B buildings (frame construction)	- 35¢
All other	- 40¢

These rates were based on a 70% co-insurance clause subject to a 1% deductible with a minimum deductible of $100 and a maximum deductible of $1,000.

In 1921 the schedule was expanded to four classes as follows:

Class A buildings	- 20¢
Class B buildings	- 30¢
Class C buildings	- 40¢
All other	- 50¢

These rates were for 50% co-insurance with credits for higher amounts as follows:

60% Co-Insurance	- 10% credit
70% Co-Insurance	- 20% credit
80% Co-Insurance	- 25% credit
90% Co-Insurance	- 30% credit

In 1923, the co-insurance credits were increased to provide a 40% credit for 90% co-insurance with corresponding increases for other amounts, and in 1925, 6 classes of construction were adopted with rate ranges in the San Francisco area of from 14¢ to 125¢.

In 1928 a minimum mandatory deductible of 10% was adopted and rates were provided for 8 construction classifications as follows:

Class	I	II	III	IV	V	VI	VII	VIII
Rate	20¢	30¢	40¢	50¢	50¢	70¢	1.50	3.50
% Deductible	10	10	10	10	10	10	15	15

In August 1928 the deductibles for Classes I, II and III were reduced to 5%.

In January 1932, the Board of Fire Underwriters of the Pacific adopted rates for an earthquake assumption endorsement which provided earthquake coverage by endorsement to a fire policy. The schedule of rates for Zone 1, which includes San Francisco and Los Angeles were as follows:

Class	I	II	III	IV	V	VI	VII	VIII
Rate	15	25	30	35	35	40	75	2.50
% Deductible	5	5	5	5	5	5	10	15

It is interesting to note that this same schedule is still in effect today although the term factor for a three year policy was increased from 2 to 2 1/2 in 1947, affording a 25% increase in rate for most policies as of that date.

The trend in stock company earthquake rates can best be described as erratic when referring to the above schedules. In 1916 it was possible to buy earthquake insurance on a steel frame building at an annual rate of 25¢ subject to 70% co-insurance and a 1% deductible. Today the annual rate in Zone 1 for this type of building is still 25¢ subject to 70% co-insurance, but the minimum deductible is now 5%. A change in the term factor has also increased the cost of earthquake insurance during this period.

The Factory Mutual Fire Insurance Companies, who are the insurers of large industrial risks throughout the United States including California, have adopted their own earthquake rating schedule. These schedules were originally adopted based on the work done by the late John R. Freeman who was for many years President of the Manufacturers Mutual Fire Insurance Company of Providence, Rhode Island, one of the Factory Mutual group. Mr. Alton C. Chick, formerly one of Mr. Freeman's principal assistants, and today Vice President and Engineer of the Manufacturers Mutual, is chiefly responsible for much of the work done in preparing and maintaining these schedules.

Space does not permit a detailed comparison of the two schedules, but it is significant to note that reductions in the cost of earthquake insurance have been made by the Factory Mutual Companies in recent years.

In addition to making decreases in the cost of earthquake insurance, the Factory Mutuals now provide earthquake sprinkler leakage coverage in California without additional cost to most policyholders. Formerly a specific charge was made for this coverage.

SUMMARY AND CONCLUSIONS

In conclusion, I would like to summarize some of the more important factors which are in need of further investigation, in the opinion of the author, in connection with the general subject of earthquake insurance in the United States.

1. Of the several specific earthquake rating factors, two are in need of further investigation and study.

   A. The area factor or the exposure to loss as a result of a single earthquake is of great importance to the underwriter. A comparison of the relative damage of earthquakes must take into account the type of construction and the type of ground upon which the building is constructed. The underwriter would be aided by more detailed information on individual earthquakes including specific information on the type of construction damaged, the type of ground upon which the building has been built, the total dollar damage, the estimate of property values in the high damage areas, thus providing a means of determining earthquake loss ratios, and the dollar damage and value of the more important buildings which have been damaged in the earthquake.

   B. With respect to the frequency of loss occurrence we need further evaluation of the relation of intensity and frequency and better analysis of damage caused by earthquakes of various intensities. The relationship of earthquake rates for various regions of the United States should be re-examined in light of such information. In this connection an up-to-date revision of "Earthquake History of the United States - Part II - Stronger Earthquakes in California and Western Nevada" based on the Modified Mercalli Scale (similar to the recent provision of Part I of this publication) would be most helpful.

2. The attitude of the insurance buyer with respect to earthquake insurance should be re-analyzed. If buyer interest indicates earthquake insurance could be sold on a much wider scale at lower rates, earthquake insurance statistics indicate rate reductions would be justified. The fact that less than 5% of the value of property insured in California against fire is also insured against the peril of earthquake may be related to the fact that earthquake rates have generally been static in that State since 1932.

It is up to the insurance industry to solve the problem of the buyer's attitude and to further investigate the credibility of statistics. The seismologist, however, can be of great assistance to the insurance underwriter and the buying public by providing better information, particularly with regard to the relationship of earthquake frequency and intensity.

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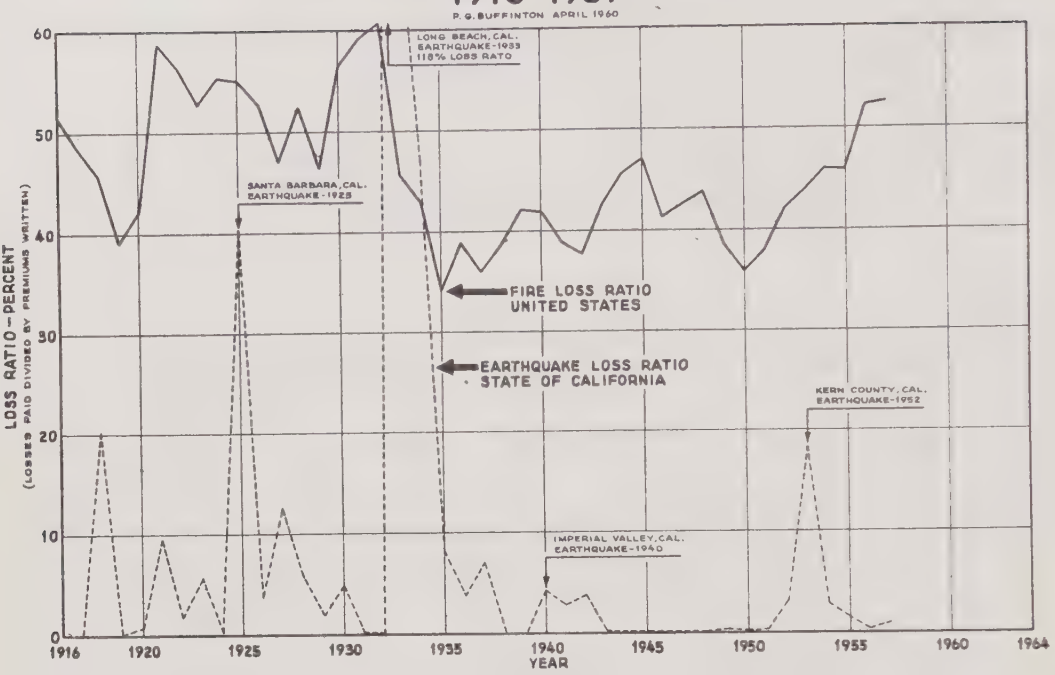
COMPARISON OF FIRE AND EARTHQUAKE LOSS RATIOS 1916-1957


P. G. BUFFINTON APRIL 1960
LOSS RATIO - PERCENT
(LOSSES PAID DIVIDED BY PREMIUMS WRITTEN)
LONG BEACH, CAL. EARTHQUAKE-1933 118% LOSS RATIO
SANTA BARBARA, CAL. EARTHQUAKE-1925
FIRE LOSS RATIO UNITED STATES
EARTHQUAKE LOSS RATIO STATE OF CALIFORNIA
KERN COUNTY, CAL. EARTHQUAKE-1952
IMPERIAL VALLEY, CAL. EARTHQUAKE-1940
YEAR

COMPARISON OF FIRE AND EARTHQUAKE WRITTEN PREMIUMS 1916-1957

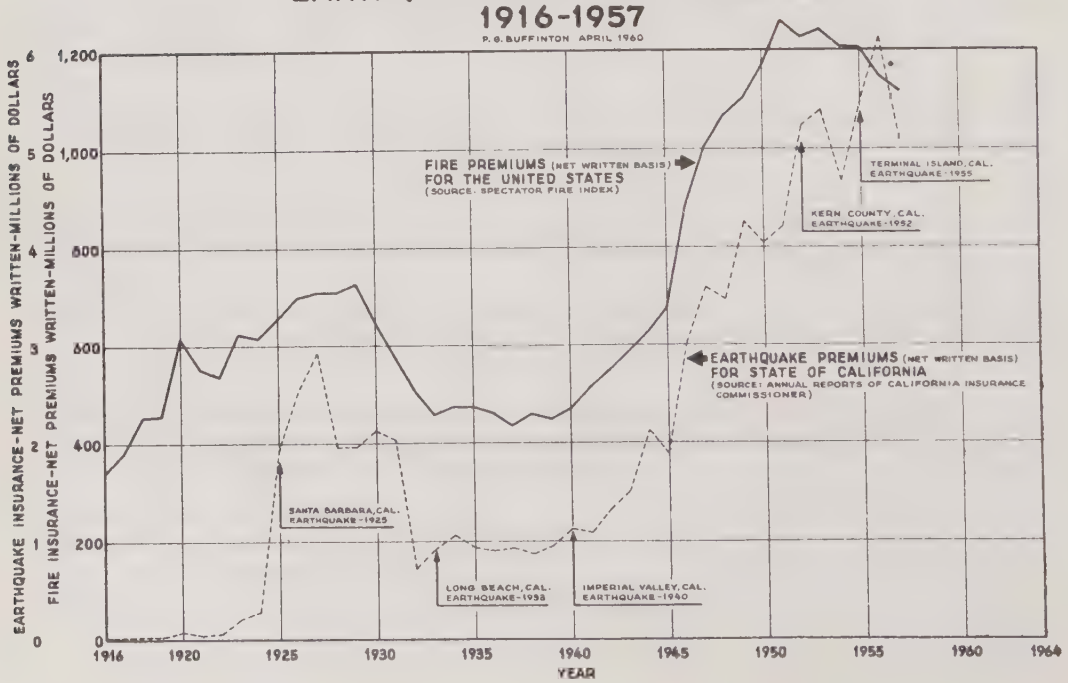

P. G. BUFFINTON APRIL 1960
EARTHQUAKE INSURANCE-NET PREMIUMS WRITTEN-MILLIONS OF DOLLARS
FIRE INSURANCE-NET PREMIUMS WRITTEN-MILLIONS OF DOLLARS
FIRE PREMIUMS (NET WRITTEN BASIS) FOR THE UNITED STATES (SOURCE- SPECTATOR FIRE INDEX)
TERMINAL ISLAND, CAL. EARTHQUAKE-1955
KERN COUNTY, CAL. EARTHQUAKE-1952
EARTHQUAKE PREMIUMS (NET WRITTEN BASIS) FOR STATE OF CALIFORNIA (SOURCE: ANNUAL REPORTS OF CALIFORNIA INSURANCE COMMISSIONER)
SANTA BARBARA, CAL. EARTHQUAKE-1925
LONG BEACH, CAL. EARTHQUAKE-1933
IMPERIAL VALLEY, CAL. EARTHQUAKE-1940
YEAR

TABLE I

FREQUENCY OF U.S. EARTHQUAKES 1638-1956

Region	Number of Years of Available Records	Intensity-Modified Mercalli Scale: 10 and over		9 and over		8 and over		7 and over	
		Total No.	Frequency	Total No.	Frequency	Total No.	Frequency	Total No.	Frequency
Northeast	318	2	159	2	159	12	26.4	25	12.7
Eastern	293	1	293	1	293	2	146.5	11	26.6
Central	257	1	257	1	257	5	51.4	24	10.7
Western Mountain	104	1	104	1	104	15	6.95	33	3.15
Wash.-Ore.	115	0	0	0	0	1	115.0	8	14.4
California	187	14	13.3	25	7.5	64	2.9	138	1.35

Notes: (1) Information from Earthquake History of the United States - Parts I and II and United States Earthquakes 1951-1956 published by U.S. Coast and Geodetic Survey.

(2) Frequency expressed as one earthquake for number of years shown.

(3) Regions as defined in Earthquake History of the United States - Parts I and II.

P. G. B.
April 1960

TABLE II

TOTAL EARTHQUAKE DAMAGE FOR EARTHQUAKES
OF VARIOUS INTENSITIES

Intensity MM Scale	Number of Earthquakes	Estimated Total Damage 1950 Dollars	Average Damage For one Earthquake
11	2	$150,000,000	$75,000,000
10-11	3	9,950,000	3,316,666
10	1	23,000,000	23,000,000
9	4	9,200,000	2,300,000
8-9	3	49,250,000	16,416,666
8	4	31,200,000	7,800,000
7	5	5,770,000	1,154,000
Summary:			
10 and above	6	182,950,000	30,491,666
8-9 and 9	7	58,450,000	8,350,000
8	4	31,200,000	7,800,000
7	5	5,770,000	1,154,000

Notes: (1) Information from "Earthquake Investigation in the United States" Special Publication No. 282 Revised (1958) Edition

(2) Damage based on 1950 evaluation of the dollar

(3) Intensities from Earthquake History of the United States - Parts I and II and United States Earthquake Series.

P. G. B.
April 1960

Session Summary

by

Hiroshi Kawasumi*

Earthquake engineering is a technical science which studies the means of aseismic design and construction of structures based on the knowledge of earthquake motions. The earthquake motion observed on the ground is subjected to the magnitude and mechanism of the earthquake at the origin, as well as to the modulation due to the length of the path and to the nature and the structure of the medium wherein the seismic waves are propagated.

Seismic regionalization or zoning, and subsoil studies are essential items in earthquake engineering. The behaviour of and damages to structures in and due to earthquakes provide us with useful knowledge for aseismic design and construction. Precise records of strong motion earthquakes and the responses of structures to known disturbances are important items of study in this field. These are included in engineering seismology, for which study seismologists engaged in earthquake engineering have hitherto concentrated their efforts.

In the present conference the fields more or less concerned with engineering seismology are dealt with in the sessions I, II, III and V.

I will here give brief summaries of the papers dealt with in the session III on Seismicity and Earthquake Ground Motion. At this session, convened in Room No. 2 on Tuesday, July 12, some miscellaneous papers were also presented, for conveniences' sake of programming, orally by the authors themselves or delivered in summarized form by a reader, together with those on Recent Strong Motion Earthquakes and the Resulting Damage.

But the papers relating to the theme of session V will not be included in this summary.

Three papers were presented on Seismicity by Hirono, Fedotov and Gajardo and Lomnitz. Into the second theme of this session, i.e. Earthquake Ground Motion, we may classify the papers of White, Kanai, Figueroa, Muramatsu and Yabashi and Sakabe. Those of Alford, Cloud and Carder, are close to this second category. The papers of Nasu, Kishinouye may also be assigned to the topics from session I. The papers of Chakravortty and Ghosh, Kumar and Buffinton may seem to have no connection with the topics of this session, but nevertheless we have no more suitable sessions for presenting of such papers. All the above papers were thus assigned by the Programme Committee of the Conference to this session. Now brief summaries of these papers will be given bellow.

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* Professor, Earthquake Research Institute, University of Tokyo.

T. Mirono read a comprehensive paper on the "Seismicity of Japan" based on the result of 37 years' routine instrumental observations made at more than 100 stations under the Japan Meteorological Agency. The mean annual frequency, during the period, of perceptible earthquakes in Japan was 1497 while major earthquakes with magnitudes over 6, 7 and 8 respectively occurred 16, 2 and 0.1 times per annum. But fortunately about ¼ of the major earthquakes had a bypocentral depth of over 60km, and 80% of the rest were of sub-oceanic origin and only minor or no appreciable damage was caused on land by the deeper or sub-oceanic earthquakes. The regional variation of seismicity is shown.

In the paper on "Seismicity of the South of the Kurile Islands" S. A. Fedotov of the Institute of the Earth's Physics in Moscow reports the results of two years' observations at six stations in the southern Kurile Islands, and concludes that the intensity on these islands, on average, will not exceed 9 degree of the Modified-Mercalli scale, even in the case of a great earthquake with magnitude over 8.

E. Gajardo and C. Lomnitz of the Institute of Geophysics and Seismology, University of Chile presented a paper on "Seismic Provinces of Chile". In this paper, the authors collected the annual frequency of reported perceptible earthquakes during 17 years in each of the 37 compartments composing of one degree of latitude. They then calculated correlation coefficients between these time series, as was done by C. Tsuboi. From the magnitude of the corralation coefficient contingency between the two compartments was inferred, and consecutive compartments with high contingencies were grouped into one seismic province. Thus they could find four such provinces as follows: A. Pampa del Tamarugal; B. Atacama Province; C. Central Chile and D. Chile Sur.

In the paper entitled "An Empirical Formula for the Spectrum of Strong Earthquake Motion" K. Kanai gave a very convenient formula derived from his theoretical studies as well as from actual observations of earthquakes in Japan and the United States of America. We can now calculate by this formula the spectrum of an earthquake motion at any point on the ground provided we assign the magnitude and epicentral distance of the earthquake and the predominant period of the ground.

I. Muramatsu and T. Yabashi deduced a similar spectrum function to that of Kanai's at a dam site based on actual observations on the spot and from a neighbouring seismic station. They used statistical data based on historical records and gave an expectancy of earthquake force as a function of the period of earthquake motion to be used as a reference in the antiseismic design of a dam to be constructed at that point.

In his paper on "The Meaning of the Spectra of Earthquake Records Obtained in or near Structures" M. P. White of the University of Massachusetts suggested an explanation for the existence of many maxima in the spectra of earthquake motions in or near buildings as the effects of vibrations from neighbouring building as well as those of the building in which the seismometers are installed. He based his explanation on the fact that forced vibration of a building by a mechanical vibrator

generated a wave in the surrounding ground which was observed as far as 750 feet with an amplitude of 0.6% of that on the roof of the building.

J. Figueroa A. of the Institute of Geophysics, National University of Mexico analysed seismograms obtained at four Mexican stations located on terrains of different geological formations. He derived the conclusions that motions with the periods of 1 sec. and 1.22 sec. predominate in earthquake motions on hard ground and in less firm ground respectively. Motions with the longer periods predominate on the surface of shore-fillings. It is also concluded that the larger are the amplitude and the duration of earthquake motion the poorer is the ground.

Although the paper on the damage of Fukui earthquake and the distructive power of similar earthquakes by Y. Sakabe of Fukui University, Japan, seems to belong to the topics of session V, it is proper to review it here because the subject of the paper is mainly concerned with the true nature of destructive earthquake motions. The author describes his personal experience, information from his friends and observations of the destructive Fukui Earthquake of June 28, 1948. This earthquake was one of the severest earthquakes ever experienced in Japan. Based on data from the above mentioned sources, the author emphasizes the importance and effects of a strong jerk in the earthquake motion in the destruction on man made structures. He refers to this sudden thrust as a shock, and suggests that this might be a result of a faulting motion which should be able to be observed in the epicentral area. The fading out of such shocks with the distance, as revealed in the seismograms at distant stations, is explained by the larger attenuation of such a large shock which he considers to be in the plastic range of deformation.

The paper of J. L. Alford of the Harvey Mudd College, Claremont, California, deals with the relation between the maximum safe charge and the distance. Examining the results hitherto published, he shows that this critical magnitude of ground motion caused by blasting is its velocity and it ranges between 4.3 and 4.7 inches per second. He recommends, allowing a margin of safety, an operating limit on the ground velocity of 2 inches per second. But in view of the fact that the ground motion caused by blasting may have a spectral intensity variation, he recommends further investigations of damage problems from blasting including the collection of information on the shape and magnitude of response spectrum as related to the weight of the explosive charge, distance and type of ground.

D. S. Carder and W. K. Cloud of the United States Coast and Geodetic Survey presented a paper on the "Ground Motions Generated by Underground Nuclear Explosions" which attracted the attention of a large audience. Carder had arrived at a formula

$$\log A = 0.75 \log W - 2.9 - \log R - 0.06 R$$

for the maximum ground displacement A (single component) in centimetre at a distance R (in km) to the explosion point where nuclear energy equivalent to that of W tons of TNT is released In the above formula,

it is assumed that the observation is made on granite, quartzite or lime-stone, but on deep alluvium the ground motion was revealed to be of a higher amplitude than at like distances on granite, the comparative figures being 2 to 3.5. He calculated the energy sent out as elastic waves at $10^{18.5}$ ergs which is equivalent to a 4.8 magnitude earthquake.

W. K. Cloud analysed 11 radial component acceleration records of the events Blanca (19 kilotons) and Logan (5 kilotons) by means of a response analyser, and he evaluated the average maximum response velocity for the oscillator period as ranging 0.1 to 2.0 sec. The plot of this value against distance showed a very uniform attenuation as the 1.78 power of the distance. These response spectra show remarkable similarity to those of natural earthquakes.

It is well-known that the intensity of an earthquake is very much influenced by subsoil conditions, and earthquake motion coming to the layered earth's surface is modulated by multiple reflections. And these motions with periods near a certain period which is characteristic of the site, and called a predominant period, are much enhanced in their magnitudes. A knowledge of this vibration characteristics of ground is very important in antiseismic problems. In the Earthquake Research Institute of Tokyo University a team concerned with subsoil research consisting of its members have been engaged in this problem and presented a part of their study in the second session of this conference under the title "Earthquake Damage and Subsoil Conditions as Observed in Certain Districts of Japan". In the present session, the former director of the Institute N. Nasu who had presided over a committee for the "Investigation of the Ground in the Northern Part of the City of Yokkaichi in the North Ise District" presented a report on the work of the committee. The items studies were 1. geology from observations on the furface and the study of 30 new boring cores and pre-existent well-logs on land area as well as in the littoral sea area; 2. subsidence of the ground surface as inferred from precise levellings; 3. soil mechanical tests: and 4. observations of micro-tremors. From these data they could classify the ground according to the building code standards, and discuss the plan of utilizing the ground. It is to be noted that the bearing power and the possible subsidence of a structure of an assigned size and weight as well as the predominant period on the ground in the area are clarified in this investigation.

F. Kishinouye of the Earthquake Research Institute also presented a paper on "Microseisms and Subsoil Conditions" in which is described a new approach to the study of subsoil conditions by means of microseisms in the frequency band between 0.1 to 0.3 sps. or so. It is shown that the distribution in a large area of amplitude ratio of microseisms observed at a spot to that at a standard station is closely related to the subsoil structures a little larger in scale than those related to micro-tremors of shorter periods.

The paper on the "Movement of Faults" of D. Tocher of the University of California at Berkeley acquaints us with interesting observation on

the progressive rupture of a winery building located on a fault line at a vineyard, seven miles south of Hollister, California. This was revealed to be due to the slow creep of the fault. The results of his precise measurement of this creep made clear that the movement of the fault is in the same direction as that observed at the time of the previous major earthquake. This study may lead to a modification of H. F. Reid's elastic rebound theory on earthquake occurrence.

"Seismological Study of the Crustal Layers in the Indian Region from the Data of Near Earthquakes" by K. C. Chakravortty and D. P. Ghosh, of Banaras Hindu University, India, contains interesting information from the seismological point of view on the thicknesses of the granitic and basaltic layers in this area of remarkable topography near the Himalayan range.

The paper of P. Kumar, of the Central Building Research Institute, Roorkee, India, on "Geotectonic Movements and their Influence on the Hydrography of the Indo-Gangetic Plains" will be of interest from the hydrographical point of view in India.

Lastly, the paper of P. G. Buffinton of the Factory Mutual Rating Bureau on "Earthquake Insurance in the United States - Reappraisal" will be reviewed. In the paper the author summarizes and analyses the development of the earthquake insurance business in the United States. which is, as is well known, a pioneer country in this field. The knowledge stated in this paper is of no small importance to the underwriters and also to the people of other earthquake countries in this field. The most of the damage incurred by severe earthquakes must be born by the people themselves. The insurance system provides for this coverage. Since the damage possibility to the properites of the people depends upon the rate of the seismicity and the liability of the structures to destruction in a strong motion earthquake. The problem in earthquake insurance must be largely solved by the progress of earthquake engineering. In conclusion of this paper, the author summarizes some of the most important factors which are needed for further investigation in connection with the general subject of earthquake insurance. The first of the factors is the area factor or the exposure to loss which depends on the seismicity and the quality of the structures, while the second is the attitude of the insurance buyer. The progress of earthquake engineering will, in the reviewer's opinion, contribute to the clarification and betterment of the above two factors. The author concludes that the seismologist can be of great assistance to the insurance underwriters and the buying public.

Although it is not necessary to say that the reviewer wants to add to the above conclusion the assistance of structural engineers in the same problem, we are happy to say that we can hope definitely for a salient development in earthquake engineering by the collaboration of the seismologists and structural engineers attending this conference. With this bright hope for a promising progress of earthquake engineering and the consequent promotion of the welfare of mankind, the reviewer would like to finish his summary.

EARTHQUAKE RESISTANT DESIGN, CONSTRUCTION AND REGULATIONS

(SESSION IV)

The questions not answered in a written form by the author do not appear on the pages of discussion.

ON EARTHQUAKE RESISTANT DESIGN OF FLAT SLABS AND CONCRETE SHELL STRUCTURES

By

Yoshikatsu Tsuboi* and Mamoru Kawaguchi**

INTRODUCTION

In the design of usual buildings, rigid frames constituted mainly of columns and beams are considered the resisting systems against external forces. Some parts of slabs and walls are very often counted as effective to cooperate with the frames. When flat slabs and shells are connected to column tops, the cooperation of these plate elements with linear members is more important than in usual framed structures. However, difficulties in dealing with these two or three dimensional elements, especially when they are submitted to partial loading, have kept a considerable part of this problem left to be solved in the future.
This paper is a description of experimental studies made to find rational design methods of earthquake resistant flat slabs (including flat plates) and concrete cylindrical shells. Scope was restricted to statics.

PART I EARTHQUAKE RESISTANT DESIGN OF FLAT SLABS

1.1 POSSIBILITIES OF EARTHQUAKE RESISTANCE OF FLAT SLABS

When the great earthquake attacked Tokyo-Yokohama area in 1923, there were already several flat slab structures in service as office buildings, warehouses and factories in these cities. We can obtain information on the degree of earthquake resistance of flat slab structures from the damage investigation reports made after the earthquake. Damage sustained by 4 flat slab buildings has been reported in detail. The flat slab buildings examined in these reports were two office buildings, a warehouse and a factory building, each having a different amount of structural walls.

Both of the two office buildings reported (five and six storied, respectively) suffered fire after the earthquake. In these buildings some cracks were found on the surface of slabs, columns and walls, and it is hard to determine whether these cracks were due to earthquake shocks or fire, but, in either case, the cracks were not fatal to the structures.

The warehouse also suffered conflagration. Reports clearly indicate that the structural damage caused by shocks was not serious and that the structure had been carrying loads on their slabs soundly until inflammable contents caught fire. According to the reports, heat generated by combustion of inflammable contents of the buildings badly weakened the concrete and reinforcement of the slabs, causing them to hang down and consequently fail.

The factory building in the Kawasaki area is the only example that did not meet fire. This three storied building had essentially no walls to bear

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horizontal forces. Condition of its damage is shown in Fig. 1. It should be noticed that vital failure occured at column tops and not in the slab. This failure could have been avoided if the column tops had been disigned with better reinforcement.

As was seen in this example, experience points out the possibilities of earthquake resistance of flat slab structures, that is, well designed flat slab structures may prove much stronger than supposed by inference from the concept of beam and column structures.

1.2 STUDIES IN THE PAST

In the earthquake resistant design of flat slabs, as in the case of usual rigid frame structures, slabs are so designed that they bear the effect of moments produced at column tops by earthquake force in addition to the effect produced by stationary loads.

The problem of elastic plates subjected to moments acting at column tops was first considered by Dr. Tanabashi[1] in 1929. Dr. Tanabashi applied Dr. Lewe's double series method[2] to this problem and examined the theoretical results through steel model tests. The author (Tsuboi) and Mr. Miyazaki[3] applied later the single series method to the same problem. Dr. Gyoten[4] also treated this problem by using the finite difference method to take rigid zone effects into account.

Above mentioned studies were confined to elastic analyses. Problems encountered in designing concrete flat slabs were treated by Messrs. Elstner and Hognestad[5] in their experimental studies on the shearing strength of reinforced concrete slabs subjected to concentrated loads. An investigation on similar problems was recently reported by Mr. Rosenthal.[6]

1.3 EXPERIMENTAL STUDIES ON FLAT SLABS SUBJECT TO COLUMN TOP MOMENT

1.3.1. Objects and Outlines of Studies

Studies reported herein were made to observe stresses and deformations in mortar slabs in elastic state to examine the approximate solutions mentioned above, and to observe general behaviors of the slabs in elasto-plastic state under monotonously increasing and reciprocally repeated loads. Both loadings were statical. Infinitely continuing flat slabs with square bays were the objects of our present studies. Specimens have beams along opposite edges, $x = \pm a$ (Fig. 3), which correspond to inflection lines of equally spanned infinitely spreading flat slabs subjected to column top bendings. This was to realize the true boundary conditions ($w = 0$, $\nabla^2 w = 0$) with tournament system supporting along these edges. Specimens were free along $y = \pm a$, the boundary conditions being different from true ones that yield $\frac{\partial w}{\partial y} = 0, \frac{\partial^3 w}{\partial y^3} = 0$, but the effect of this difference is considered secondary. Specimens were made of mortar, strength of which being nearly equal for all slabs. Loading mechanism used is shown in Fig. 2. Moment was statically applied through the column stub while reactions were made to occur at four equally spaced points along the inflection lines through tournament systems.

Three of the nine specimens were made of plain mortar (FO-1, FO-2 &

FO-3). The rest were reinforced in three different ways, that is, reinforcement in the direction of the x - axis (the direction included in the plane in which the column stub moment is applied) was apportioned more to the column strip than to the intermediate strip for F1-1 & F1-2, uniformly distributed for F2-1 & F2-2 and apportioned less to the column strip than to the intermediate strip for F3-1 & F3-2 (Fig. 3). One of the two specimens having identical reinforcement distribution was subjected to monotonously increasing moment, while the other was subjected to reciprocally repeated moment. In the latter case, loading was returned at three stages; first little outside the elastic region, then in elasto-plastic state and finally very near the ultimate strength. Strains at various locations on slab surfaces and the angle of rotation of column stubs were measured.

1.3.2. Test Results in Elastic State

a. Stress Distribution:

As strains were measured in a quarter part of each test slab, symmetry of loading regarding the x - axis is essential to have the results in this part represent the whole slab.
This was checked by the strains at the points of the opposite portion with respect to the x - axis. In the test of FO-2 (plain mortar) the best symmetry was achieved (difference of strains of two opposite points was less than 7 per cent), so strain measurements of FO-2 were used for comparison with theoretical approximations. Similar strain distributions were recorded for the other slabs.

Calculations were made by the two approximations for a slab identical in dimension with those tested; one of these approximations was single series method in which loads distributed over the column top areas to represent applied external moments were expanded in Fourier's series, and the other was finite difference method in which perfectly rigid zones were assumed over the column top areas.

Distributions of $\overline{M}_x = -D\frac{\partial^2 w}{\partial x^2}$ obtained by these two methods are shown in Fig. 4. Comparison with the test results shows that either method is practically sufficient to express stress distributions. Ratios of longitudinal moments apportioned to column and intermediate strips are as shown in Fig. 4.

b. Stiffness — Effective Width:

When interaction of columns and slabs is to be considered in flat slab structures, the conception of effective width -- width of an imaginary beam having the same depth, span and stiffness against rotation of the column as those of the slab under consideration -- is introduced.

As stated above, introduction of rigid zones around the column tops is not necessarily essential to obtain stresses in the slabs with practically sufficient accuracy. However, it can be easily supposed that the effect of rigid zones is important for the estimation of effective stiffnesses. This effect was examined as follows.

In single series method, if we have the longitudinal tangent of the de-

flection surface at the column top center represent the angle of rotation of the column top,

$$\theta = \frac{\partial w}{\partial x}\Big|_{\substack{x=0\\y=0}} = 0.0625\,\frac{M}{\pi^5 R^4 D}. \tag{1}$$

This gives the following effective width for an imaginary beam with no rigid zone for poisson's ratio $\nu = 1/6$.

$$2b_e' = \frac{M}{Eh^3\theta}(2a) = 0.672\,(2a). \tag{2}$$

When translated for a beam, for comparison, with a perfectly rigid zone for $-c \leqq X \leqq c$, this gives

$$2b_e = 2b_e'\left(\frac{a-c}{a}\right)^3 = 0.345\,(2a). \tag{3}$$

On the other hand, finite difference method in which perfectly rigid zone is assumed over the column top area gives, for $\nu = 1/6$,

$$2b_e = 0.620(2a) \tag{4}$$

Test results are as shown below. In calculating effective width from observed angle of rotation of column stubs, effect of reinforcement was taken into consideration for the cross section as a whole, and the difference in distributions of reinforcement was not considered. Results show that the latter effect is not appreciable within the elastic region. Effective width thus obtained by the tests is

$$2b_e = 0.58 \sim 0.61\,(2a), \tag{5}$$

showing a good agreement with that obtained by finite difference method assuming perfectly rigid zone.

1.3.3. Test Results in Elasto-Plastic State and Ultimate Strength

As the behavior of plain mortar slabs is of little interest to us from the view-point of earthquake resistance, detailed description is omitted. Notable tendancies are; though failure (bending) was visually quite sudden with no cracks found before it, strain measurements show them spreading from the center towards edges, making slabs fail at about half of ultimate loads obtained through beam tests by assuming the slabs also to be beams. Moment - Strain and Angle of Rotation Curves of F0-2 are shown in Fig. 5.

Six of nine test slabs were reinforced in three different ways, though the total cross sectional area of reinforcement was kept the same for all slabs. All of them were doubly reinforced, upper and lower reinforcings being the same. For test slabs F1-1 and F1-2, 70 % of the total longitudinal reinforcement was distributed within column strips. For F2-1 and F2-2 the longitudinal reinforcement was uniform over the whole section. For F3-1 and F3-2 only 30 % of the longitudinal reinforcement was arranged in column strips, the rest being distributed in intermediate strips.

In the tests of F1-1, F2-1 and F3-1, column moment was monotonously increased up to failure, while in the case of F1-2, F2-2 and F3-2, it was statically repeated three times in different stages as shown by an example for F1-2 in Fig. 8.

Moment-Angle of Rotation (of column stub) curves of F1-1, F2-1 and F3-1 are shown in Fig. 7 by solid lines and virgin curves for F1-2, F2-2

and F3-2 by dotted lines. It should be noticed that in both loadings, distribution of reinforcement has considerable effects on the slab stiffness in elasto-plastic region.

Of six reinforced slabs, only F1-1 which had denser reinforcement in its column strip and was monotonously loaded failed in pure flexure. On the other hand, only F3-2 which had lighter reinforcement in its column strip and was repeatedly loaded, failed in punching. For the remaining slabs, two kinds of failures apparently coexisted. As shown in Fig.9 pattern of punching observed in the test is of a different nature from that experienced in usual flat slab tests made without or with little column moment. Punching took place very gradually and no sudden decrease of load was observed. Crack patterns for a few slabs are shown in Fig. 6.

Ultimate moments of test slabs are indicated in Fig. 7. Ultimate moment of F1-1 which failed in pure flexure is in good agreement with that calculated by the yield line theory, taking into consideration the effect of strain hardening of reinforcement, and confirmed for the present case by beam tests. As to F2-1 & F3-1,and F1-2 & F2-2 in positive loading, yielding was spreading when punching began, and for F3-2,and F1-2 & F2-2 in negative loading, punching strength was reached before yielding began.

Two tendencies are seen with respect to the ultimate strength of test slabs: Distribution of longitudinal reinforcement affects the punching shear resistance around the column; and repeated load reduces the punching shear resistance, especially after the foregoing opposite load reached near the ultimate load.

PART II EARTHQUAKE RESISTANT DESIGN OF CONCRETE CYLINDRICAL SHELLS

2.1 GENERAL VIEW

As shell roofs are comparatively new types of structures, we do not have sufficient knowledge about how they behave under earthquake shocks. As one of the most popular kinds of shell roofs, concrete cylindrical shells are taken up here.

Cylindrical Shells are often constructed on columns, and it is usual for traverses or stiffners to cross the sections at the locations of columns. Sometimes they are also provided with edge beams in the longitudinal direction.

As we lack the knowledge concerning the earthquake resistance of shell roofs, cylindrical shell roofs are usually so designed that they may receive no moments from the column tops, by making or assuming pin joints at the connections with columns. Consequently, columns must be designed as cantilevers clamped at the bases. However, this is by no means rational nor economical. Design methods by which the cooperation of shell slabs and edge beams can be properly evaluated are required.

When the horizontal component of earthquake force is transversely applied, stresses in a shell due to body force may be of some importance in some cases[6], but, as long as the effect of column top moment is concerned,

a traverse with an effective fraction of the shell does not essentially differ from a beam of the usual frame structure.

On the other hand, when longitudinal moments are applied at column tops, shell slabs seem to lend more effective cooperation to beams, and they will play quite an essential role when no edge beam exists. Thus it is the longitudinal column top moment that particularly draws our attention.

2.2. STUDIES IN THE PAST

The above mentioned problem in the elastic region was considered by Mr. Heki[9] who applied Dr. Zerna's approximation,[10] which is essentially identical to the second approximation of ASCE Mamual No. 31, by expanding antisymmetrically distributed line loads at column tops in Fourier's series. The same problem was recently treated by Mr. Okamoto.[11]

2.3. EXPERIMENTAL STUDIES ON CYLINDRICAL SHELLS SUBJECT TO COLUMN MOMENT

2.3.1. Objects of Studies

In the analysis of a cylindrical shell subject to longitudinal earthquake force, the column top moment is first replaced by a line load distributed on the column top, which is then expanded in trigonometric series to give longitudinal boundary conditions to the shell.

From the standpoint of design, however, this approach has its faults that the convergence of series is not rapid, especially when the ratio of the longitudinal width of column top to the longitudinal span is small.

In the case of flat slabs, the problem is of course similar, but column width - span ratio is comparatively large. On the contrary, in cylindrical shells, large column width is usually considered unnecessary as long as stationary loads are concerned, because internal forces of the shells are transfered to columns chiefly through traverses. Thus the above ratio often falls on a value less than 1/10. Moreover, calculation involved is much more laborious in this case than in flat slabs.

Objects of our studies were to examine the validity of elastic approximation, to find simpler and more practical ways to obtain stresses and the effective stiffness of the shell, and to observe the general behavior of concrete shells (mortar models were used here) in its elasto-plastic state.

2.3.2. Parallel Shells

a. Celluloid model test:

To reproduce the phenomenon which would take place when parallel continuous shells are subjected to moments at equally spanned column tops, a celluloid specimen shown in Fig. 10 was made and moment was applied through the centrally located column.

This model has only two spans in the transverse direction. As it is expected that stresses and deformations rapidly decrease with the distance

from the column, this kind of model may be used to represent infinite parallel shells. The longitudinal edges of the model were supported by celluloid diaphragms, and, as in case of flat slab tests, inflection lines were assumed there. Longitudinal span was varied as shown in Fig. 10. Strains at various points, deflection of the longitudinal column line and rotation of the column stub were measured.
Results of the tests were as follows.

Stresses:

Transverse bending moment M_φ and longitudinal stress σ_x obtained from strains observed are indicated in Fig. 11 for the case where L = 17 cm. Results of the series method based on Zerna's approximation are shown in the same figure for comparison. First 6 terms were taken in the above calculation. Convergence of σ_x was not very good yet, but the both stresses are seen in fairly good agreement with those observed. For L = 43.8 cm, distributions of σ_x on a few transverse measuring lines are shown in Fig. 12 compared with those calculated by the beam method. It can be seen that observed σ_x approaches that obtained by the beam method when the distance from the column increases. It was also observed that considerable changes in longitudinal span produces little changes in stresses around the column.

Effective Stiffness:

Rotation of the column stub measured by the mirror method is plotted in Fig. 13 for various longitudinal spans. It can be seen that the straight line applied to the observed angles of rotation of comparatively long specimens ($L \geq r/2$) is parallel to the line obtained by the beam method.

The test results mentioned above can be reduced to the following statement concerning the nature of comparatively long parallel shells subject to column top moments: Deformations of the shells are chiefly governed by beam action, but disturbance takes place around the columns. This disturbance diminishes rapidly with increase of the distance from the column, and it seems that stress distributions around the column have little sensitivity to the changes in longitudinal span in the case of comparatively long shells. As it will be compared later with that of single shells, this is a nature peculiar to parallel shells. This characteristic helps us make an engineering approach to the present problem. As stated above, convergence of series is generally very slow in long shells. In such cases, shorter shells may be considered instead and the results can be modified by the beam method. This nature also simplifies the way of thinking when we try to find effective stiffnesses of parallel shells. If we try to assume the effective width in shells as in flat slabs, we encounter the difficulty that there are too many factors which affect the results.
In the case of flat slabs these factors are the shape and size of the bay and the column top section. When both the bay and the column top section are square, the only variable is the column top width - span ratio. For cylindrical shells, two variables must be added to the general case of flat slabs, that is, radius of the section and thickness of the shell slab. Thus, at least four variables, the ratios of longitudinal and transverse spans, shell slab thicknesses and longitudinal column top widths, to radius of the section, for instance, have to be involved in the expression of an effective width. On the other hand, taking advantage of the characteristic

of parallel shells, we can separate the longitudinal span from the above variables for comparatively long shells. Then column stub rotation is considered a sum of that due to beam deformation and that due to partial deformation around the column, the latter being independent of the longitudinal span.

Thus the problem can be simplified to some extent and a more rational approach to the practical solution seams to be possible by this method than by the conception of effective width.

b. Mortar model tests

Eight models shown in Fig. 14 were made of reinforced mortar and subjected to repeated column top moments. They have various rises (f/b = 0, 0.05, 0.10, 0.20) and one of the two specimens which have the same rise has, and the other does not have, an edge beam. The loading system used in flat slab tests was again employed without any change in principle. The test results were as follows.

Stresses:

As it was difficult to fabricate specimens with constant and uniform thicknesses, and as values of strains were small, we could not expect sufficient accuracy in observed strains. However, the following information was obtained: Prominence of beam action is common for all specimens. Edge beam restrains partial deformation around the column and diminution of local stresses with increase of the distance from columns was seen as well as in specimens with no edge beams. Increases in rise also decrease stresses in shell slabs, but the influence of this change in rise was not very large.

Stiffness:

Observed angle of rotation of column stub is plotted in Fig. 15 with curves obtained by the beam method shown for comparison. When f = 0 (flat), as stated in the description of flat slab tests, rigid region of column top area restrains the rotation of column stub, and observed rotation of both (with and without edge beam) specimens were much less than expected by the beam method. With increase in rise, this effect becomes less, and conversely, the effect of local deformation becomes prominent. Fig. 15 also suggests that a change in rise does not have a large effect on local deformation.

Cracks and Failure:

All specimens were subjected to reciprocally repeated loads. The method of loading was the same as that used for flat slab tests. Two flat specimens failed in bending, and, as they were sufficiently long, behaved completely like beams. In shell specimens, initial cracking took place at around 20 % of ultmate load. In the case of specimens without edge beam, first cracks appeared on the upper surface along the longitudinal column line. This is apparently due to M_φ which shows high concentration around the column in the theoretical solution, too. However, this crack did not extend long,nor was it fatal. In the case of those with edge beams, initial cracking occured at the point of contact of column and beam.

For all of the shell specimens, most prominent and vital were shear cracks which spread from the corner of columns at approximately 45 degrees to the generatrix. All of the shells failed by local failure around the columns.

In Fig. 16, applied ultimate moments of shell specimens are plotted with calculated maximum moments of the section (single span) for comparison. It can be seen from the figure that S1-1 and S1-2 would develop the maximum strength of their sections in actual structures. Examples of crack patterns are shown in Fig. 17.

2.3.3. Single Shells

Three single shell specimens as shown in Fig. 18 were made of reinforced mortar. All of them have edge beams. The feet of the specimens were so pinned that horizontal reactions at these four points will be equal, and a reciprocally repeated horizontal load was statically applied at the level of column tops. Deflections of shell surfaces and strains at various locations were measured.
Test results were as follows.

Deflections and Stresses:

Deflection surfaces of the three specimens are shown in Fig. 18 for the same load. There is a great difference between the deflections of flat and shell specimens, both in magnitude and in character. But there is no appreciable difference between those of shell specimens with different rises (f = 0.10b & 0.20b). The same thing can be observed regarding the stresses.

Concerning the stress distributions, another characteristic of single shells is that the beam method can not be applied. Distribution of longitudinal direct stresses at a section near the column is characterized by Fig. 19. Effective stiffness is much less than expected by the beam method (1/8 ~ 1/10).

Therefore, the concept of beam deformation plus local deformation is not suitable for single shells. For these, the conception of effective width is preferable for the estimation of effective stiffnesses.

Cracks and Failure:

Crack pattern is different from that of parallel shells. As transverse edges largely deflect over the longitudinal span, cracks appear along these edges, but do not spread much toward the top of the shells. Crack patterns of shell specimens are shown in Fig. 20. Failure occured at the connection of shells and columns. Mode of failure was complicated.

SUMMARY

Preliminary studies for rational design methods of earthquake resistant flat slabs and concrete cylindrical shells were described in this report.

Attempts were made to grasp the general nature of their behaviors under column top moments. For both structures, series methods can be applied to obtain stresses, but for the estimation of effective stiffnesses, either some modification of the methods or application of other methods by which the effect of rigid zone can be introduced is required. In the practical design of parallel long shells, slow convergence of series can be avoided by making use of the prominence of beam action in these structures. Concept of effective width is suitable for the practical design of flat slabs and single shells, but in the case of parallel shells, introduction of beam action will give simpler and more rational design methods. In the placement of reinforcement of flat slabs, not only stress distribution in elastic region, but also the influence of reinforcement on stiffness in elasto-plastic region and the ultimate strength should be considered. Further studies are required to learn the nature of punching of flat slabs under column top moment. In parallel shells, shear strength of shell slabs is important, while the most prominent stresses in elastic solutions are longitudinal direct stresses and transverse moments. In single shells deformation is very large, and shells can not be expected to stiffen edge beams to a great extent. Cracks appear over a wide range. Strength of connection is essential.

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Fig. 1

Example of Damage

Fig. 2-1

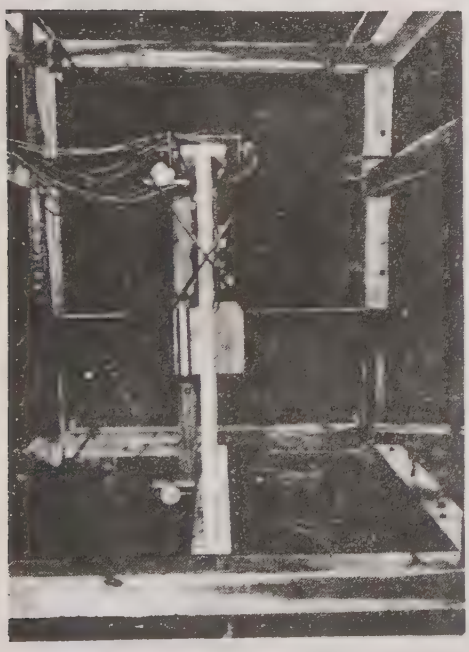


Fig. 2-2

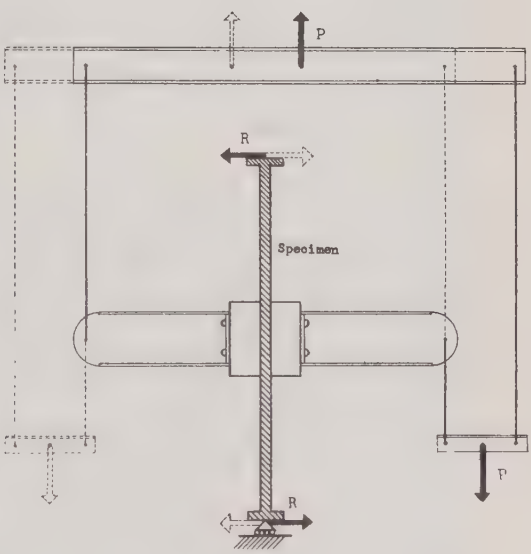


Fig. 2 Loading System

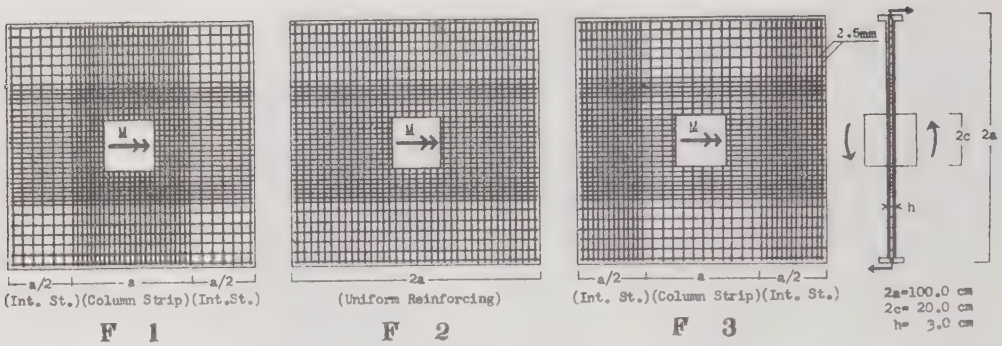


Fig. 3 Flat Slab Specimens

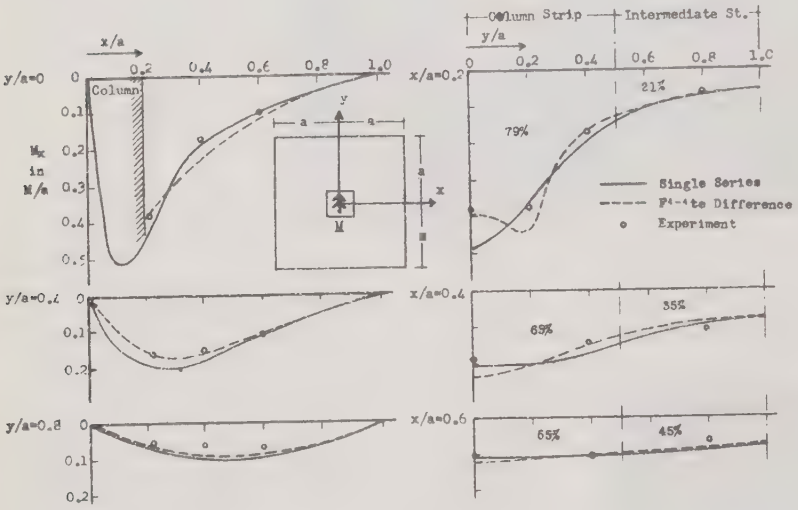


Fig. 4
Distribution of $\bar{M}_x$

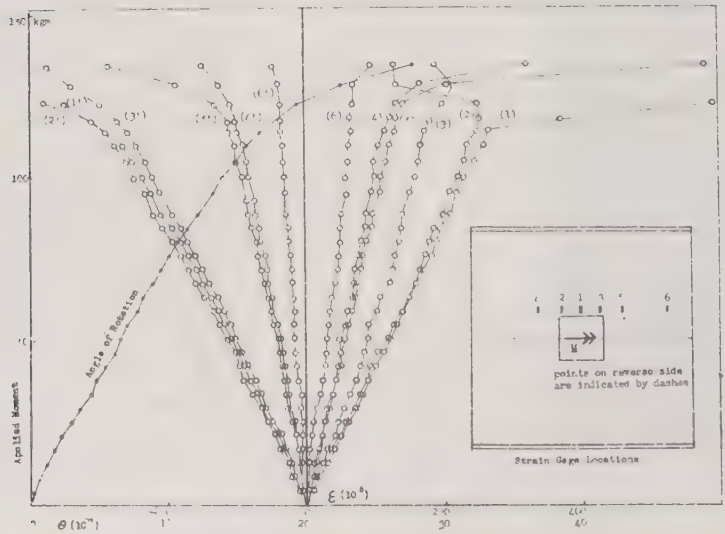


Fig. 5
M-ε
M-θ
Curves

Fig. 6 Crack Patterns at Failure

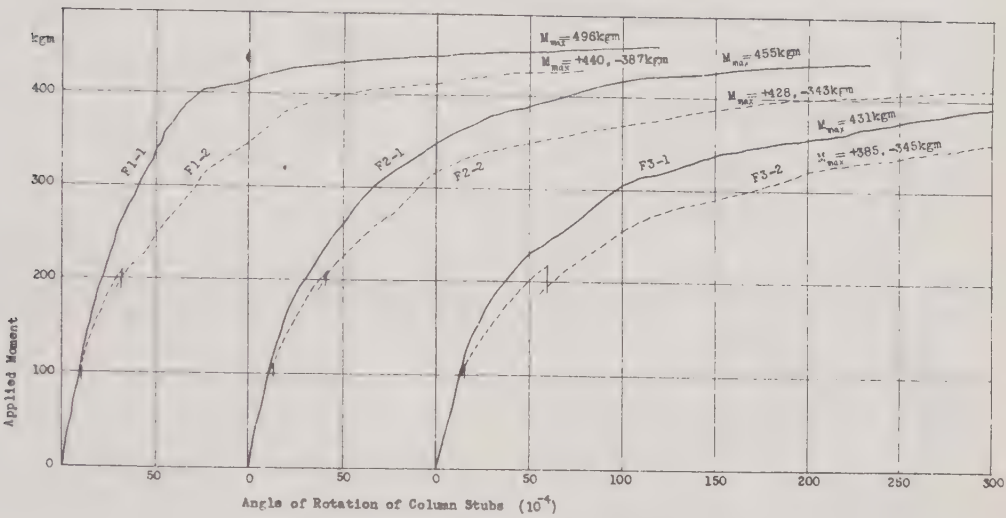


Fig. 7 M-θ Curves

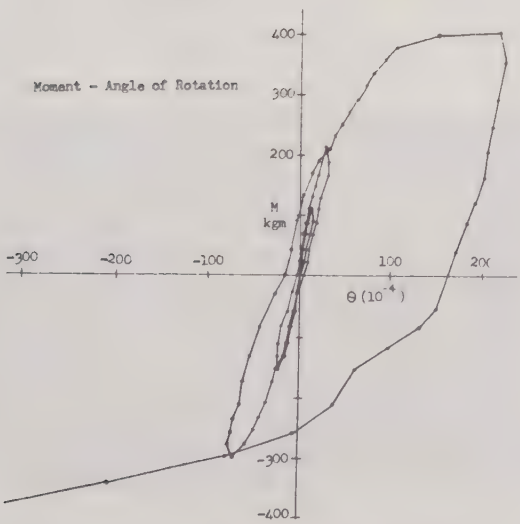


Fig. 8 M-θ Curve under Repeated Load

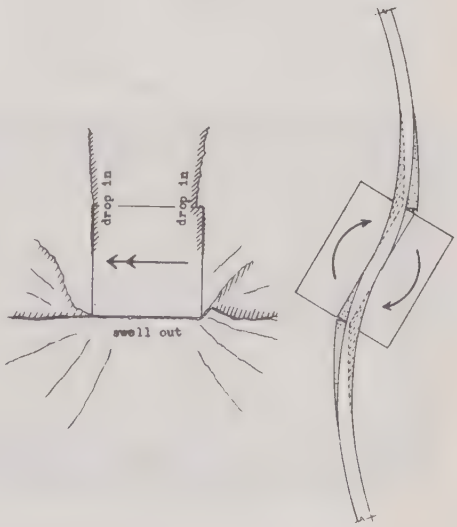


Fig. 9 Punching Pattern

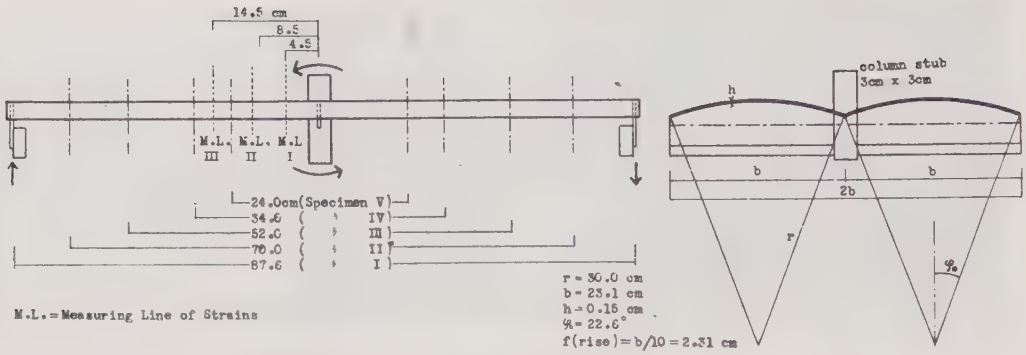


Fig. 10 Celluloid Specimens

Fig. 12 σ_x

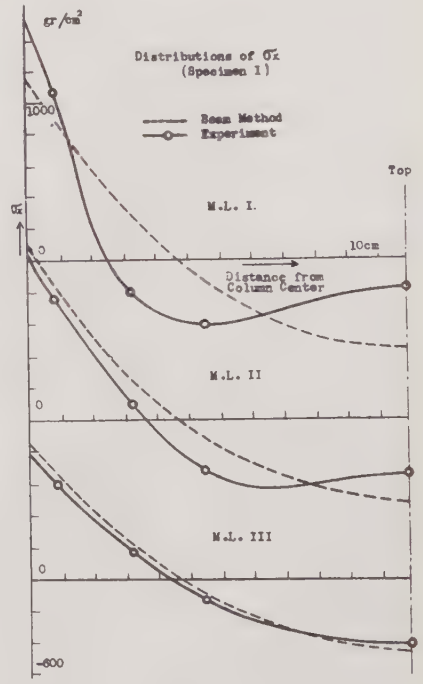
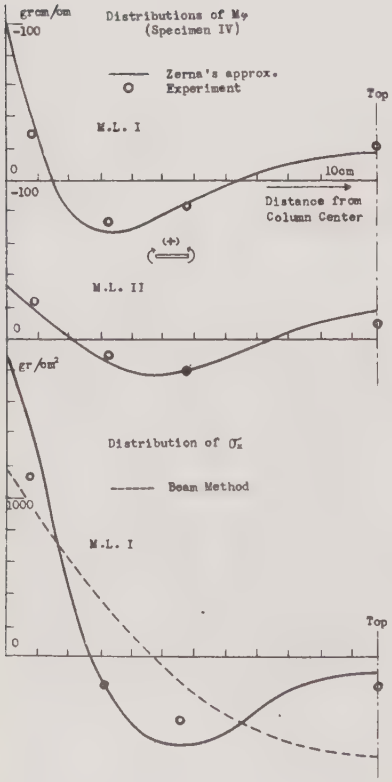


Fig. 11 M_φ & σ_x



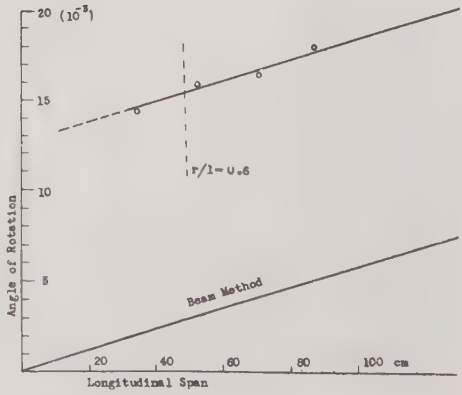
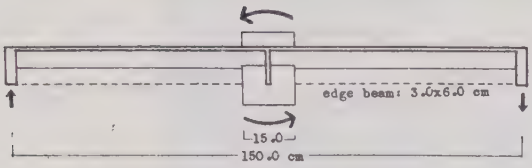


Fig. 13 Rotation-Span



Specimens

f/b	no edge beam	with edge beam
0	S 0 - 1	S 0 - 2
0.05	S 1 - 1	S 1 - 2
0.10	S 2 - 1	S 2 - 2
0.20	S 3 - 1	S 3 - 2

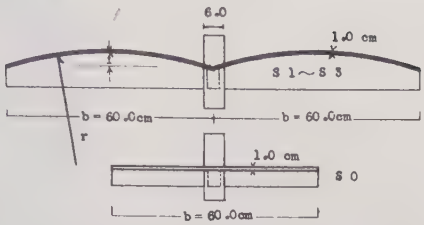


Fig. 14 Parallel Shell Specimens (Mortar)

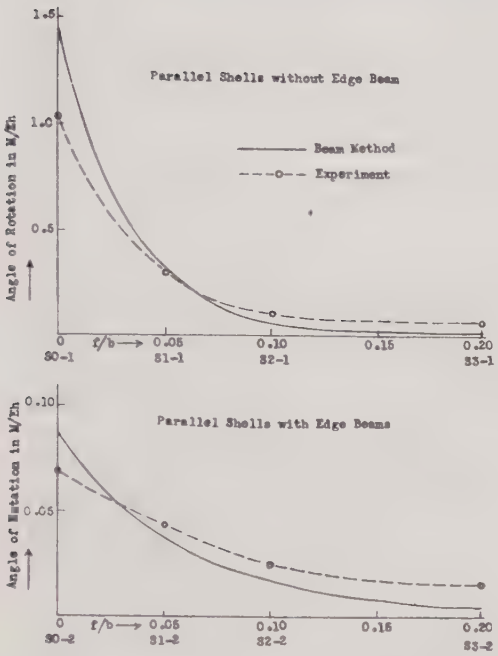


Fig. 15 Angle of Rotation

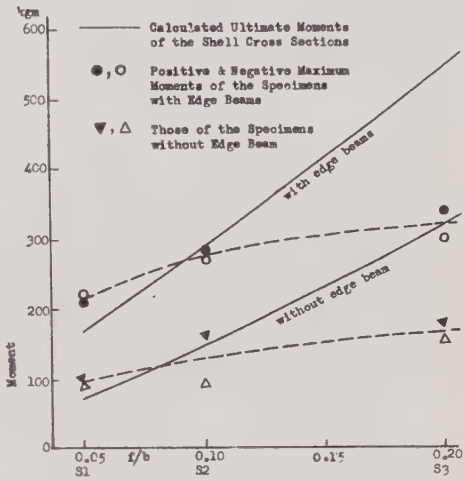


Fig. 16 Ultimate Load

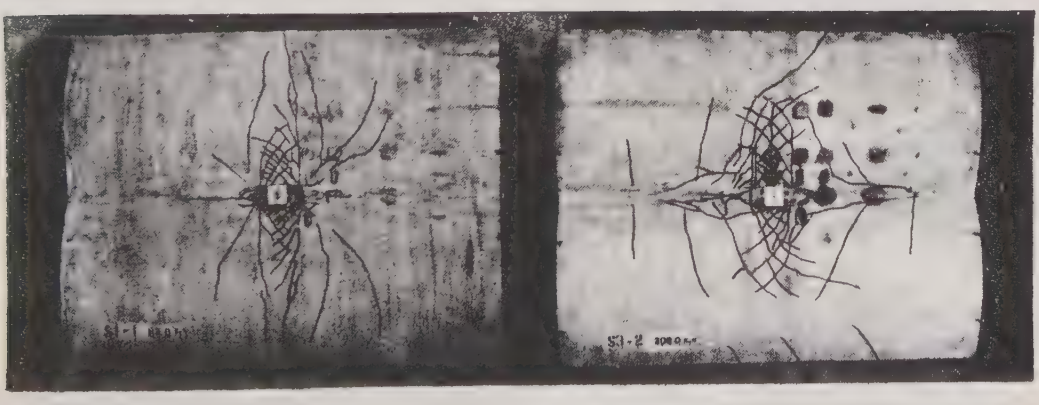


Fig. 17 Crack Patterns

Fig. 18-1

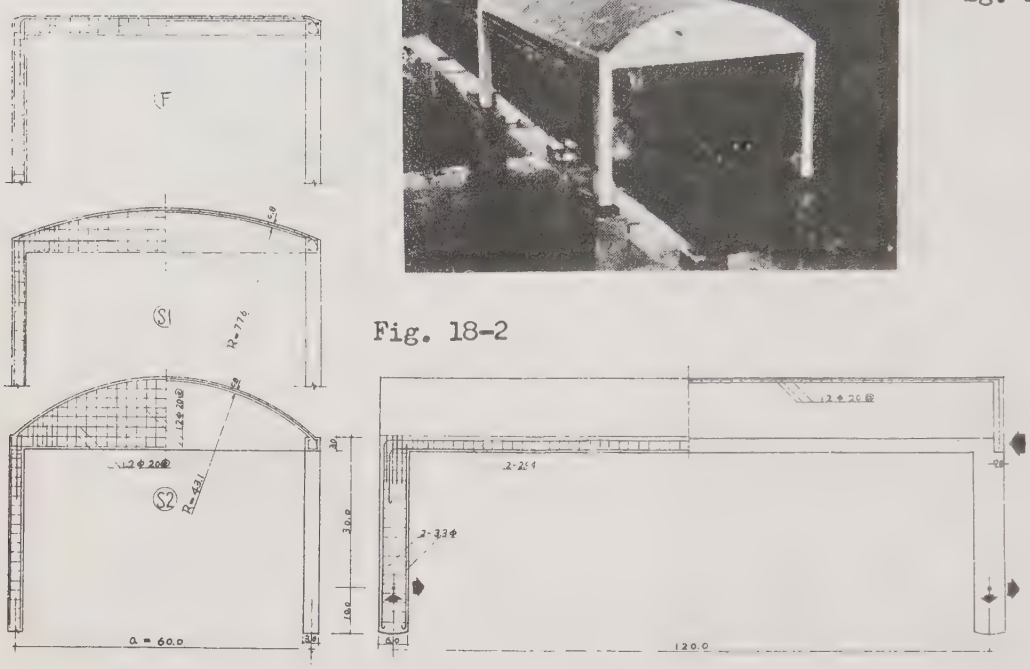


Fig. 18-2

Fig. 18 Single Shell Specimens

Fig. 19 Deflections

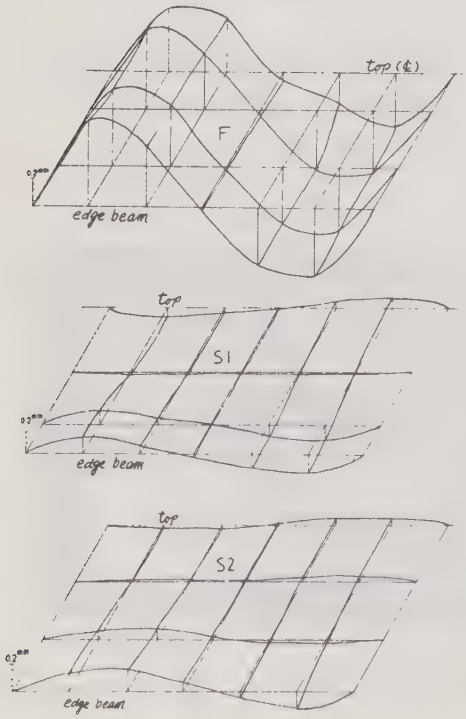


Fig. 20 Longitudinal Stress

Fig. 21 Crack Patterns

DESIGN CRITERIA FOR SHEAR AND OVERTURNING MOMENT

by John E. Rinne*

Introduction

Criteria for earthquake-resistant design in California have varied considerably over the fifty-four years since the San Francisco Earthquake of April 18, 1906. In the early post-quake era, earthquake resistance was achieved through design for increased wind forces. While some structural engineers designed for accelerations applied to gravity loads, it was not until 1933 that earthquake design was written into the State laws. The Riley Act provides for minimum lateral earthquake forces of 2 per cent of the vertical design loads. The Field Act, also passed in 1933, following the Long Beach Earthquake, is applicable to schools and other institutional buildings under the jurisdiction of the State Division of Architecture. The rules established under this Act are much more stringent than the Riley Act. The latter is generally applicable statewide where more stringent requirements are not prescribed by local ordinances. Most of the cities and counties do impose much more involved design criteria in their building codes.

As this is being written, the Los Angeles and Uniform Building Codes are considering adopting the recommendations of the Seismology Committee of the Structural Engineers Association of California. These recommendations are the subject of another paper at this Conference. All of the modern codes have recognized the need for variable coefficients for seismic shears, with higher coefficients for low, rigid buildings; lower coefficients for high, flexible buildings. In the Los Angeles and Uniform Building Codes, these coefficients relate to the number of stories above the story under consideration and are applied to the dead load generally to arrive at the story earthquake shears. Overturning criteria correspond to 80 per cent of the story shear multiplied by the height to the center of gravity of the loads above the story under consideration.

Following the report of the Joint Committee on Lateral Forces (San Francisco Section, American Society of Civil Engineers, and Structural Engineers Association of Northern California) known as "Separate 66", San Francisco adopted a code where, for the first time in California at least, earthquake forces and shears were directly related to the fundamental period of vibration of the structure. The overturning moments were recognized not to be cumulative without limit. An arbitrary limit on design overturning moment was given:

> "Provision for overturning moment shall be made for the specified earthquake forces in the top ten stories of buildings or the top 120 feet of other structures, and the moments shall be assumed to remain constant from these levels into the foundations."

The "overturning moment" at any horizontal plane is the moment on the structure as a whole resulting from the dynamic earthquake forces above the plane, giving due regard to signs of the modal forces.

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The Joint Committee was well aware that this was an interim provision and recommended that further research was needed for "establishment of a design criterion for the overturning effects in earthquakes, based upon dynamic considerations."

The author undertook a study of shears and overturning moments of stack-like structures and more recently, as part of the work of the Seismology Committee of the Structural Engineers Association of California, extended this work to include shear deflection type structures. From these studies, criteria have evolved for design overturning moment. It is the purpose here to present the reasoning leading to these criteria.

Steps in the Analysis

To correlate the ensuing discussion, the following outline of the steps in the development may be helpful toward an understanding of the significance of these steps and of the assumptions made.

1. Determine equivalent spring-mass systems representative of the modes of vibration for uniform cantilevers which deflect (a) due to bending moments exclusively, and (b) due to shear exclusively.

2. Establish a modal response spectrum which indicates the response of the simple spring-mass systems to earthquake ground motion in terms of relative lateral force, or base shear, on these structures as a function of the period of vibration.

3. Determine the combined modal response of both moment and shear type structures, and for both base shear and base moment. Compare the base moment response with the base moment corresponding to the SEAOC code provisions for base shear and overturning corresponding to that base shear.

4. Indicate a criterion for reducing the base moment to reflect the ratio of the base moment response to the base moment corresponding to the design base shear.

5. Recommend a basis for reducing the design overturning moment from the base to the top of the structure.

6. Recommend a basis for distributing the design overturning moment to the vertical resisting elements of the structure.

Equivalent Spring-Mass Systems

The response of a complex structure to earthquake ground motion theoretically can be determined at any instant during an earthquake by the superposition of the responses in the different modes in which the structure can respond or vibrate. Each of these modes can be represented by a spring-mass system of period equal to that of the mode under consideration, and a mass which is a certain proportion of the total mass of the structure.

For cantilever structures of uniform distribution of mass or weight, Mr. Milton Ludwig has determined these equivalent spring-mass systems for

moment deflecting structures and for shear deflecting structures. With his permission, Mr. Ludwig's development is given in detail in the Appendix. The resulting equivalent masses of the modal spring-mass systems, together with the ratios of the higher mode periods to the fundamental period, are given in TABLE I. Graphically these are shown in Figures 1 and 2 which indicate heights of the spring-mass systems such that the modal shears multiplied by these heights give the respective modal base overturning moments in the actual structure.

It is seen by the summation of W_e/W columns of TABLE I that most of the mass of either type structure, but particularly the shear type, is assigned to the first three modes. For purposes of relating design shear to design overturning moment, a simplifying assumption can be made that all of the mass is assigned or prorated to the first three modes. The higher modes contribute little to overturning and even the third mode is not very significant in this regard. The result is shown in TABLE II and in Figures 1 and 2.

Response Spectrum

The spectral or maximum responses of spring-mass systems to a number of recorded ground motions have been determined for both undamped and damped systems, using analog computers. It bears repeating that the spectral response is the maximum response during the entire earthquake. As units of measurement, the spectra may be in terms of displacement, velocity, acceleration, or functions related to these units. For this analysis, the acceleration spectrum is particularly useful since this spectrum is directly related to the lateral force, or the base shear, on the spring-mass system.

Reviewing a number of undamped spectra, which qualitatively can indicate the response of structures within the elastic range, the striking similarity between these is interesting. A response spectrum which seems to envelop quite adequately the spectra from a number of earthquakes of varying intensity is shown in FIGURE 3.

An assumption will be made in this analysis which is admittedly not entirely accurate. It is that spectral responses can be added to get the combined modal response. The maximum responses in the second and third modes do not necessarily occur at the same instant as the spectral fundamental response. But it is probable that the higher modes are making a significant contribution at that instant. The accuracy of this assumption is probably well within the accuracy of several other assumptions inherent to this kind of an analysis.

Combined Modal Responses

The relationship between base moment and base shear in the first three modes is shown in TABLE III. The calculation of the modal base moments and the ratio of these to the moments corresponding to the design base shear is given in TABLE IV for moment structures, and TABLE V for shear structures. These calculations have been made for certain periods selected either arbitrarily or to create the maximum response in the second or third modes. For example, $T_1 = 1.56$ seconds was selected to give a second mode period, $T_2 = 0.25$ for the moment type structure, at which period the second mode response is a maximum as seen from FIGURE 3.

Moment Reduction Criterion

The results of TABLES IV and V are plotted in FIGURE 4. Also shown thereon is the proposed base moment reduction factor, J.
FIGURE 2 indicates that if the base shear for design is given by a formula:

$$V = C\,W, \quad \text{where } C = k/\,T^{1/3}$$

and if this base shear is resolved into forces F_x by the formula:

$$F_x = V \frac{w_x\, h_x}{\Sigma\, w\, h}$$

and if the modal response to earthquake ground motion is given by FIGURE 1:

$$S = 0.25/\,T \quad \text{for T greater than or equal to } 0.25$$

$$\text{and} \quad S = 0.30 + 2.80\,T \quad \text{for T between 0 and } 0.25$$

and if it is a reasonable approximation to assume that the maximum response of the structure is the sum of the spectral modal responses, then the design base moment M can be given by:

$$M = J \Sigma F_x\, h_x$$

$$\text{where } J = J_b \text{ for buildings} = 0.5/\,T_1^{2/3} \quad \text{with } J_{b\,max.} = 1.00$$

$$\text{and } J_{b\,min.} = 0.33$$

$$\text{and } J = J_m \text{ for moment deflection structures, like stacks}$$

$$J_m = 0.6/\,T_1^{1/2} \quad \text{with } J_{m\,max.} = 1.00$$

$$J_{m\,min.} = 0.40$$

The J_m factor is in the same form as the factor k suggested in the author's earlier paper, but is somewhat lower.

$$k = 0.7/\,T_1^{1/2} \quad \text{with } k_{max} = 1.00$$

$$k_{min} = 0.50$$

The difference is attributable to the difference in the criteria for the base shear coefficient C. Here the formula advocated by the Seismology Committee of the Structural Engineers Association of California has been used. Earlier, the formula for $C = 0.08 / T_1$.

It would have been possible to fit empirical curves much closer to the response curves in FIGURE 2. A degree of conservatism has been introduced to account for the possibility that the modal response might peak at a period somewhat higher than 0.25 seconds. Most of the spectra peak in the range of 0.20 to 0.30 seconds. The recommended formula for J recognizes this contingency.

Base Shear

The SEAOC Seismology Committee's recommendation for base shear is:

$$V = K C W \quad \text{where} \quad C = 0.05/T^{1/3}$$

and K = 0.67 to 1.33 for buildings, depending upon the type of construction

K = 1.50 for structures other than buildings.

While the recommended code places no lower limit on the value of C as given by the formula above, it does place a ceiling or maximum value of C as that corresponding to T = 0.10 seconds. This is C = 0.108. One and two story buildings have a fixed C = 0.10. A C = 0.10 maximum for all structures is a simpler and more consistent upper limit. Depending upon the type of structure and construction, the combined K C factor then would vary from 0.067 to 0.15. For the base shears so determined, the base moment M can be defined by the formulas given above.

The Moment M_x

The variation of the overturning moment on the structure as a whole over the height of the structure depends upon the relative importance of the fundamental and higher modes of vibration. This could lead to a quite involved relationship which hardly seems justified. For a moment deflection structure of the uniform cantilever type it has been shown that a straight-line diminishing moment from the bottom to the top of the structure provides a reasonable design criterion. For more rigid structures, where the fundamental mode predominates, this straight-line variation is conservative. However, recognizing that overturning considerations do not usually impose difficulties to the design of low structures, and in the interest of simplification, it is suggested that the moment M_x at height h_x be given by:

$$M_x = M \left(1 - \frac{h_x}{H}\right)$$

The code further recommends or stipulates that the total moment at any level x shall be assigned to the vertical resisting elements in the same proportion as the shears. Further amplification of this is to be found in the manual which explains the code in more detail.

Conclusion

It is believed that the SEAOC criteria for shear and overturning moment are more refined and more realistic than like criteria used heretofore. However, it must be acknowledged that experience in applying these criteria to date have been very limited. Hence, it might be expected that further modifications in the code will be forthcoming with further experience. For now, the criteria appear to be a sound step forward in the resolute progress of earthquake-resistant design.

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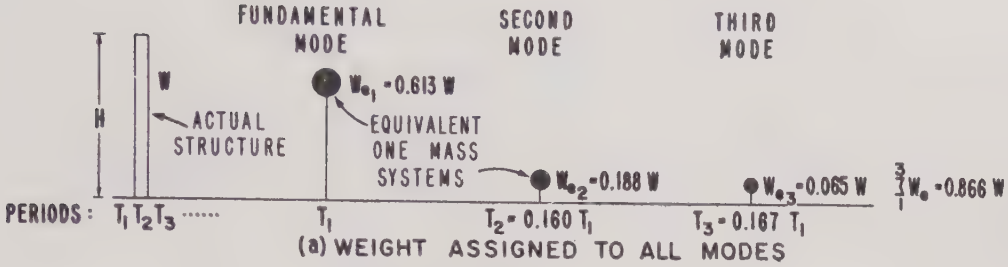
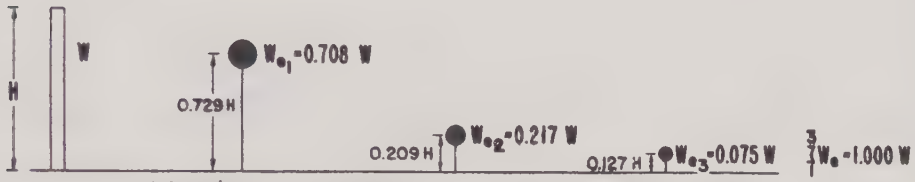
Nomenclature

C	Base shear coefficient in $V = K C W$
F_x	Earthquake lateral force at level x
h_x	Height to level x from the base of the structure
H	Total height of the structure from the base
J	Base moment reduction coefficient
J_b	J as applied specifically to buildings and structures other than those structures which deflect primarily due to bending, like stacks.
J_m	J as applied to moment deflection type structures
K	Coefficient for type of structure and type of construction in determining the base shear, $V = K C W$
M	Base overturning moment for design; also the base moment response in Tables IV and V
M_d	Base moment corresponding to $\sum F_x h_x$
M_x	Overturning moment for design at level x
S	Relative spectral response of spring-mass systems
T_n	Period of vibration in mode n
V	Base shear; also the base shear response in Tables IV and V
V_d	Base shear corresponding to $C = k/ T^{1/3}$ in Tables IV and V
w_x	That part of the total weight of the structure which is at or is assigned to level x for purposes of lateral force design
W	Total weight of the structure
W_{en}	Proportion of the total weight W of a uniform cantilever assigned as an equivalent spring-mass system for mode n.



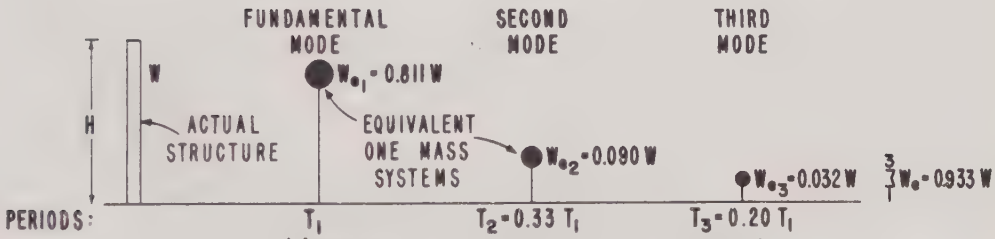
(a) WEIGHT ASSIGNED TO ALL MODES



(b) WEIGHT ASSIGNED TO FIRST THREE MODES

EQUIVALENT SPRING — MASS SYSTEMS FOR MOMENT DEFLECTION STRUCTURES

FIGURE 1



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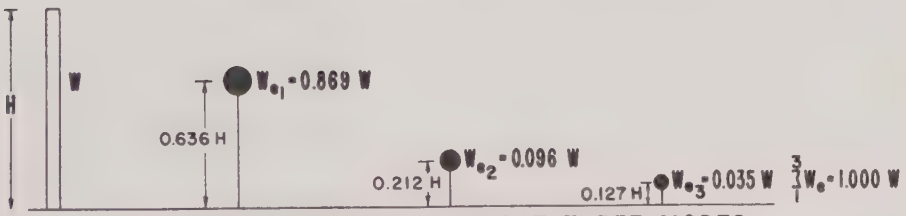

(b) WEIGHT ASSIGNED TO FIRST THREE MODES

EQUIVALENT SPRING — MASS SYSTEMS FOR SHEAR DEFLECTION STRUCTURES

FIGURE 2

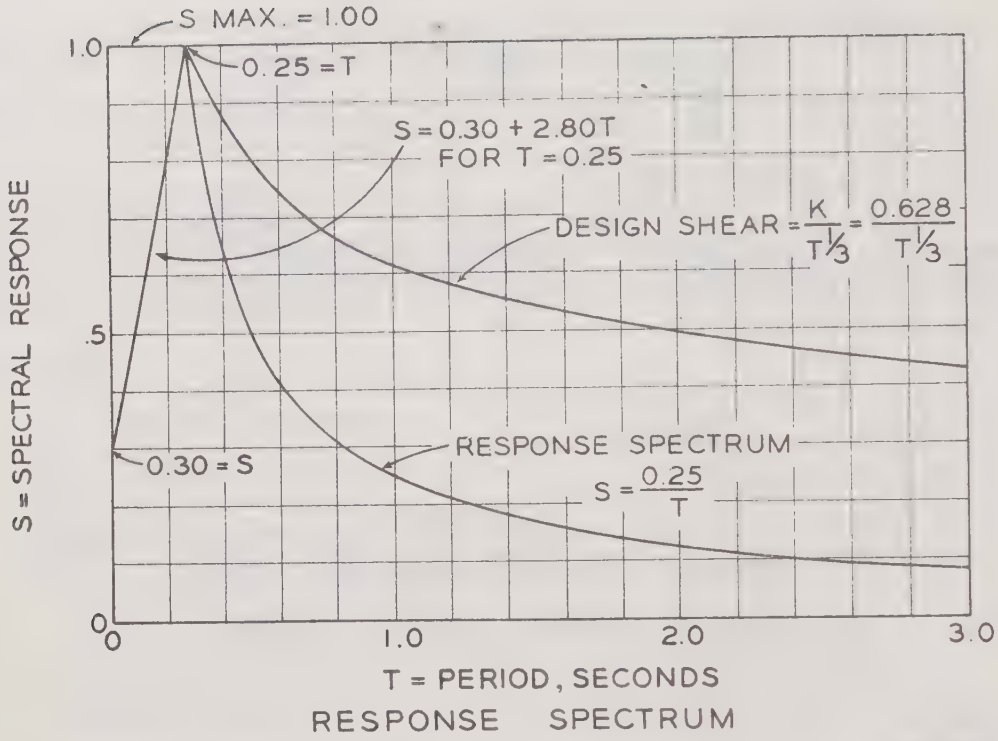

FIGURE 3

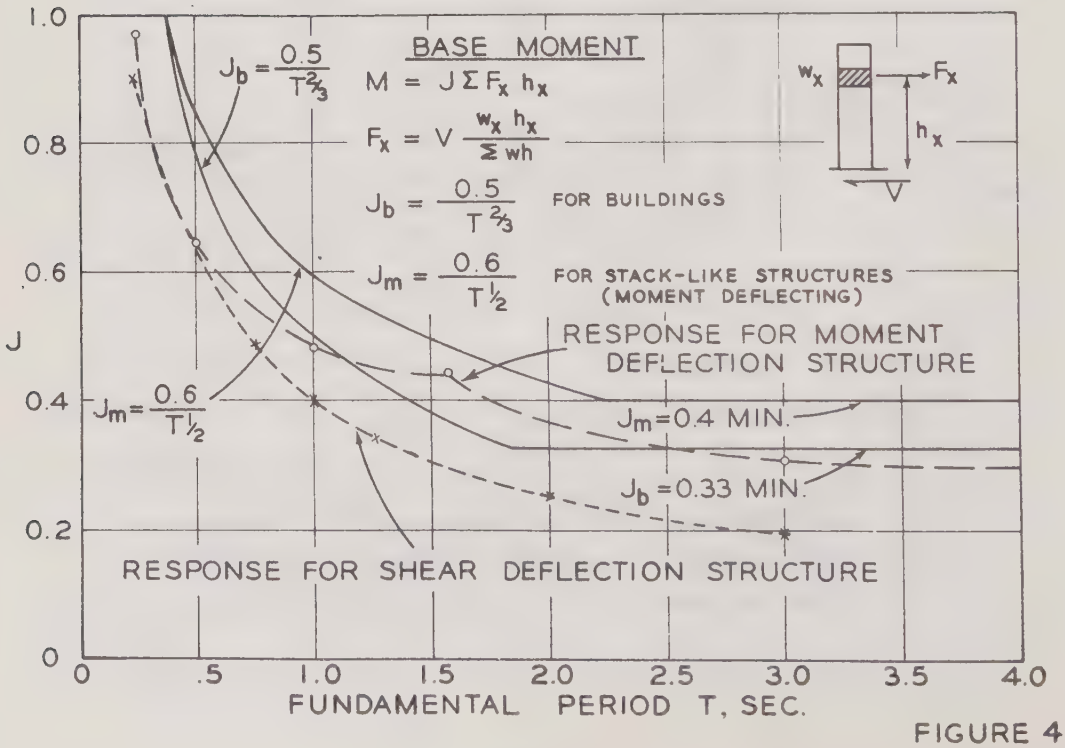

FIGURE 4

TABLE I
EQUIVALENT MODAL SPRING-MASS SYSTEMS*

Mode	Moment Deflection Structures			Shear Deflection Structures		
n	W_e/W	$\Sigma W_e/W$	T_n/T_1	W_e/W	$\Sigma W_e/W$	T_n/T_1
1	0.613	0.613	1.00	0.811	0.811	1.00
2	0.188	0.801	0.160	0.090	0.901	0.33
3	0.065	0.866	0.057	0.0324	0.933	0.20
4	0.033	0.899	0.029	0.0166	0.950	0.142
5	0.020	0.919	0.0176	0.0100	0.960	0.111
6	0.013	0.932	0.0117	0.0067	0.967	0.091

*For cantilevers of uniformly distributed weight.
W_e/W = proportion of total weight assignable to the indicated mode's equivalent spring-mass system.
T_n/T_1 = ratio of the modal period to the fundamental period.

TABLE II
PROPORTIONATE ASSIGNMENT OF ALL WEIGHT TO MODES 1, 2, 3

Mode	Moment Deflection Structures			Shear Deflection Structures		
n	W_e/W	$\Sigma W_e/W$	T_n/T_1	W_e/W	$\Sigma W_e/W$	T_n/T_1
1	0.708	0.708	1.00	0.869	0.869	1.00
2	0.217	0.925	0.160	0.096	0.965	0.33
3	0.075	1.000	0.057	0.035	1.000	0.20
	1.000			1.000		

TABLE III
BASE MOMENT - BASE SHEAR RELATIONSHIPS

Mode	Moment Structures M/VH*	Shear Structures M/VH
1	0.729	0.636
2	0.209	0.212
3	0.127	0.127

*When H = 1, M/VH = M/V

TABLE IV
MOMENT RESPONSE FOR MOMENT DEFLECTION STRUCTURES

Item	Description	Fundamental Period, T_1					
		0.25	0.50	1.00	1.56	3.00	4.38
S_1	Response	1.00	0.50	0.25	0.16	0.085	0.057
W_{e1}	Equiv. mass	0.708	0.708	0.708	0.708	0.708	0.708
V_1'	$S_1 W_{e1}$	0.708	0.354	0.177	0.113	0.060	0.0403
T_2	0.160 T_1	0.040	0.080	0.160	0.250	0.480	0.703
S_2	Response	0.42	0.523	0.746	1.000	0.52	0.355
W_{e2}	Equiv. mass	0.217	0.217	0.217	0.217	0.217	0.217
V_2'	$S_2 W_{e2}$	0.091	0.114	0.162	0.217	0.114	0.0771
T_3	0.057 T_1	0.014	0.0285	0.057	0.089	0.171	0.250
S_3	Response	0.33	0.38	0.46	0.55	0.78	1.00
W_{e3}	Equiv. mass	0.075	0.075	0.075	0.075	0.075	0.075
V_3'	$S_3 W_{e3}$	0.025	0.0285	0.0345	0.0412	0.0586	0.075
V'	$V_1' + V_2' + V_3'$	0.824	0.4965	0.3735	0.3712	0.2326	0.1924
V_1	$V_1'/ 0.824$	0.860	0.430	0.215	0.1375	0.0731	0.0490
V_2	$V_2'/ 0.824$	0.111	0.138	0.196	0.263	0.139	0.0932
V_3	$V_3'/ 0.824$	0.029	0.0343	0.042	0.050	0.0715	0.0910
V	$V_1 + V_2 + V_3$	1.000	0.6023	0.453	0.450	0.2386	0.2332
M_1	0.729 V_1	0.6270	0.3140	0.1570	0.1010	0.0533	0.0358
M_2	0.209 V_2^2	0.0232	0.0289	0.0410	0.0550	0.0291	0.0195
M_3	0.127 V_3^2	0.0038	0.0043	0.0053	0.0063	0.0091	0.0115
M	$M_1 + M_2 + M_3$	0.6540	0.3472	0.2033	0.1623	0.0915	0.0668
V_d*	Design shear	1.000	0.794	0.628	0.541	0.436	0.386
M_d	0.667 V_d	0.667	0.530	0.420	0.361	0.291	0.258
J	M/ M_d	0.980	0.655	0.484	0.449	0.314	0.259

$$* V_d = \frac{K}{T_1^{1/3}} = \frac{0.628}{T_1^{1/3}}$$

TABLE V

MOMENT RESPONSE FOR SHEAR DEFLECTION STRUCTURES

Item	Description	Fundamental Period, T_1					
		0.25	0.75	1.00	1.25	2.00	3.00
S_1	Response	1.00	0.33	0.25	0.20	0.125	0.093
W_{e1}	Equiv. mass	0.869	0.869	0.869	0.869	0.869	0.869
V_1'	$S_1 W_{e1}$	0.869	0.290	0.217	0.174	0.1085	0.0721
T_2	0.333 T_1	0.083	0.250	0.333	0.417	0.667	1.00
S_2	Response	0.532	1.00	0.757	0.600	0.375	0.250
W_{e2}	Equiv. mass	0.096	0.096	0.096	0.096	0.096	0.096
V_2'	$S_2 W_{e2}$	0.0510	0.096	0.0728	0.0576	0.0360	0.0240
T_3	0.200 T_1	0.050	0.150	0.200	0.250	0.400	0.600
S_3	Response	0.44	0.72	0.86	1.00	0.625	0.416
W_{e3}	Equiv. mass	0.035	0.035	0.035	0.035	0.035	0.035
V_3'	$S_3 W_{e3}$	0.0154	0.0253	0.0301	0.0350	0.0219	0.0146
V'	$V_1' + V_2' + V_3'$	0.9354	0.4112	0.3199	0.2666	0.1664	0.1107
V_1	$V_1'/0.9354$	0.929	0.310	0.232	0.186	0.116	0.077
V_2	$V_2'/0.9354$	0.055	0.102	0.0775	0.062	0.0386	0.0256
V_3	$V_3'/0.9354$	0.016	0.0268	0.0321	0.0374	0.0232	0.0155
V	$V_1 + V_2 + V_3$	1.000	0.4388	0.3416	0.2854	0.1778	0.1181
M_1	0.636 V_1	0.5910	0.197	0.148	0.118	0.074	0.049
M_2	0.212 V_2	0.0117	0.0216	0.0164	0.0131	0.0082	0.0054
M_3	0.127 V_3	0.0020	0.0034	0.0041	0.0047	0.0029	0.0020
M	$M_1 + M_2 + M_3$	0.6047	0.2220	0.1685	0.1358	0.0851	0.0564
V_d *	Design shear	1.000	0.692	0.628	0.584	0.498	0.436
M_d	0.667 V_d	0.667	0.462	0.420	0.390	0.333	0.291
J	M/M_d	0.907	0.480	0.401	0.348	0.255	0.194

* $V_d = \frac{0.628}{T_1^{1/3}}$

APPENDIX 1

Development of Equivalent Spring-Mass Systems for Uniform Cantilever Beams of the Moment Deflection Type

(From notes prepared by Mr. Milton Ludwig, Engineering Department, Standard Oil Company of California, San Francisco, California)

The modal periods of vibration of a uniform cantilever beam deflecting due to bending moment is well documented as:

$$T_n = \frac{2\pi}{\theta_n^2}\sqrt{\frac{W H^3}{E I g}} \qquad \text{.....(1)}$$

where θ_n is defined by:

$$\cos\theta_n \cosh\theta_n + 1 = 0 \qquad \text{.....(2)}$$

The spring constant and mass for the equivalent spring-mass systems may be evaluated in several ways. One method is to determine the effect on frequency for a slight lateral flexibility at the base. An easier and more accurate method is to work from the known response to a sudden change of lateral velocity at the base. The solution to this problem has been given* and with some changes in notation, the equation for the modal shear is:

$$V_n = B_n v\sqrt{\frac{E I W}{H^3 g}}\ \sin\omega_n t \qquad \text{.....(3)}$$

where V_n = base shear in the nth mode

B_n = numerical constant for the nth mode

v = base velocity

W = total mass of the beam

ω_n = frequency of nth mode, radians per sec.

t = time, sec.

H = total height of cantilever

EI = Young's modulus x moment of inertia

From equation (1) for small values of t:

$$\sin\omega_n t = \omega_n t = \theta_n^2\sqrt{\frac{E I g}{W H^3}}\ t \qquad \text{.....(4)}$$

which substituted into equation (3) yields:

$$V_n = B_n v \theta_n^2 \frac{E I}{H^3} t \qquad \text{.....(5)}$$

The spring constant is defined as the force-deflection ratio, so:

$$k_n = \frac{V_n}{v\,t} = B_n \theta_n^2 \frac{E\,I}{H^3} \qquad \text{.....(6)}$$

The basic equation for the resonant frequency of a spring-mass system may be used to give the equivalent mass for each mode, as follows:

$$W_{en} = \frac{k_n\,g}{\omega_n^2} = W \frac{B_n}{\theta_n^2} \qquad \text{.....(7)}$$

in which W_{en} = mass associated with the nth mode

The report* that provides the solution for V_n (Equation 3) also gives the equation for the moment developed at the base of the cantilever for a sudden change of velocity:

$$M_n = A_n\,v\,H\sqrt{\frac{E\,I\,W}{H^3\,g}} \qquad \text{.....(8)}$$

Then the ratio of the moment arm to the total height of the beam is:

$$\frac{M_n}{H\,V_n} = \frac{A_n}{B_n} \qquad \text{.....(9)}$$

Values of A_n and B_n and calculated equivalent masses and moment factors are as follows:

Mode, n	θ_n	A_n	B_n	$\frac{W_{en}}{W} = \frac{B_n}{\theta_n^2}$	$\frac{M_n}{H\,V_n} = \frac{A_n}{B_n}$
1	1.875	1.566	2.146	0.610	0.729
2	4.694	0.868	4.149	0.188	0.209
3	7.855	0.509	3.994	0.065	0.127
4	10.996	0.364	4.000	0.033	0.091
5	14.137	0.283	4.000	0.020	0.071
6	17.279	0.231	4.000	0.013	0.058

These calculated values are perfectly general and are not limited to the case for a sudden change of velocity. The values of $M_n/H\,V_n$ can be calculated from the moment and shear equations for harmonic vibration of a cantilever beam, and equivalent masses can be checked by other methods. It was merely convenient to use the constants from the NACA report which seem to have been worked out carefully and accurately.

*Report No. 828 National Advisory Committee for Aeronautics (NACA) "Bending and Shear Stresses Developed by the Instantaneous Arrest of the Root of a Moving Cantilever Beam." by Elbridge Z. Stowell, Edward B. Schwartz, and John C. Houbolt. 1945. Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. Price 15 cents.

APPENDIX 2

Development of Equivalent Spring-Mass Systems for Uniform Cantilever Beams of the Shear Deflection Type

(From notes prepared by Mr. Milton Ludwig, Engineering Department, Standard Oil Company of California, San Francisco, California)

In this development:

M = total mass of the shear deflection type structure

M_1, M_2, etc. = masses of the equivalent spring-mass systems, corresponding to the fundamental, second, mode etc.

K = total spring constant for the structure $= \frac{\text{force at top}}{\text{deflection at top}}$

k_1, k_2, etc., = spring constants for the equivalent spring-mass systems

ω = angular velocity of an applied simple harmonic base motion

ω_1, ω_2, etc., = angular velocity corresponding to the fundamental, second mode, etc.

$v = v_{max} \sin \omega t$ = applied lateral velocity at the base

V = periodic base shear resulting from the applied velocity

$Z = V/v$ = input impedance of the system

$j = \sqrt{-1}$, which merely indicates the phase relationship between the base shear V and the velocity v.

L = inductance of an equivalent electrical system or analog.

C = capacitance of the electrical analog.

It can be demonstrated that the structure involved here is equivalent to a long short-circuited transmission line with no resistance.

$\frac{M}{g} = L$ = equivalent total inductance of the structure

$\frac{1}{K} = C$ = equivalent total capacitance of the structure

for which $Z = j\sqrt{\frac{L}{C}} \tan \omega\sqrt{LC} = j\sqrt{\frac{M K}{g}} \tan \omega\sqrt{\frac{M}{K g}}$

For the equivalent spring-mass systems:

$$Z = j \frac{\omega M_1/g}{1 - \omega^2 \frac{M_1}{k_1 g}} + j \frac{\omega M_2/g}{1 - \omega^2 \frac{M_2}{k_2 g}} + \cdots\cdots\cdots$$

The impedance Z is infinite when $\frac{M}{K g} = \pi/2, \quad 3\pi/2, \quad 5\pi/2$, etc.

or when $\omega\sqrt{\frac{M_1}{k_1 g}} = 1, \quad \omega\sqrt{\frac{M_2}{k_2 g}} = 1, \quad$ etc.

from which, $\frac{M_1}{k_1} = \left(\frac{2}{\pi}\right)^2 \frac{M}{K}, \quad \frac{M_2}{k_2} = \left(\frac{2}{3\pi}\right)^2 \frac{M}{K}$, etc.

Now, the response at a frequency very close to a resonant frequency is very high so that the response of the other masses of the equivalent system can be neglected. So to determine M_1:

$$\tan \omega\sqrt{\frac{M}{K g}} = \frac{1}{\tan \frac{\pi}{2}\left(1 - \frac{2}{\pi}\omega\sqrt{\frac{M}{K g}}\right)} = \frac{1}{\tan \frac{\pi}{2}\left(1 - \frac{\omega}{\omega_1}\right)}$$

in which $\omega_1 = \frac{\pi}{2}\sqrt{\frac{K g}{M}}$ = first resonant frequency

As $\frac{\omega}{\omega_1}$ approaches 1, $\tan \omega\sqrt{\frac{M}{K g}}$ approaches $\frac{1}{\frac{\pi}{2}\left(1 - \frac{\omega}{\omega_1}\right)}$

For the shear deflection structure: $Z = \frac{j\sqrt{\frac{M K}{g}}}{\frac{\pi}{2}\left(1 - \frac{\omega}{\omega_1}\right)}$

For the first mass of the equivalent spring-mass system:

$$Z = \frac{j\omega \frac{M_1}{g}}{1 - \frac{\omega^2 M_1}{k_1 g}} = \frac{j\omega \frac{M_1}{g}}{1 - \left(\frac{\omega}{\omega_1}\right)^2} = \frac{j \frac{\omega}{\omega_1} \frac{M_1}{g} \frac{\pi}{2} \frac{K g}{M}}{\left(1 + \frac{\omega}{\omega_1}\right)\left(1 - \frac{\omega}{\omega_1}\right)}$$

Then equating values of Z:

$$M_1 = \frac{4}{\pi^2} \frac{1 + \frac{\omega}{\omega_1}}{\frac{\omega}{\omega_1}} M = \frac{8}{\pi^2} M = 0.811\ M$$

as $\frac{\omega}{\omega_1}$ approaches 1

$$k_1 = \frac{\pi^2}{4} \frac{M_1}{M} K = 2K$$

By an exactly analogous development:

$$M_2 = \frac{1}{9} \frac{8}{\pi^2} M = 0.090\ M$$

$$M_3 = \frac{1}{25} \frac{8}{\pi^2} M = 0.0324\ M$$

$$M_4 = \frac{1}{49} \frac{8}{\pi^2} M = 0.0166\ M$$

ASEISMIC DESIGN OF TWO LONG-SPAN MULTI-STORY BUILDINGS

*Dimitry Vergun and David V. Whitmore**

INTRODUCTION

The problem of designing any multi-story building in a seismic region is a serious one, and should be approached with caution. Codes exist which guard against negligence on the part of the engineer and builder and serve as a minimum standard and guide. It must be kept in mind, however, that codes are based to a large extent on experience with existing structures. When considering the design of a structure which departs from conventional practice, engineering judgment would tend to require a more thorough investigation of the effects of earthquakes than may normally be necessary or justified. The most obvious manner in which to do this is to investigate the seismic history of the area in which the proposed structure is to be built and establish the effect of probable earthquakes on the proposed structural system.

A rational basis for this procedure is as follows: Thanks to the U.S. Coast & Geodetic Survey there are available numerous seismographs of strong-motion earthquakes recorded along the Pacific Coast. Many of these records are for earthquakes of magnitude 7 and over, which locally did damage to buildings corresponding to intensities up to VIII and IX. Although none of the records represent the maximum earthquake which some seismologists believe to be possible (in view of the strength of the earth's crustal mantle generally believed to be approximately M-9), some are rather close in magnitude and intensity to the 1906 San Francisco Earthquake (8.3).

It was felt by the authors that in view of the moderate magnitude of most Pacific Coast earthquakes and the relative infrequency of earthquakes 8.0 and over that a structure would perform satisfactorily if designed to withstand with little or moderate damage all moderate shocks and at least one major shock (8.0 or over) in its lifetime. The structure should also be designed so that in an earthquake of catastrophic proportions it would not imperil life, even though structural damage would be unavoidable.

DESIGN NO. I: THE NORTON BUILDING

The Norton Building, designed by Bindon & Wright, Architects, and Skidmore, Owings & Merrill, Consulting Architects and Engineers, is a 21-story office structure (Fig. 1), located in Seattle, Washington, in a region of moderate seismic activity. Between 1930 and 1951 there have been in Seattle approximately 25 earthquakes of intensity greater than MM III, of which two were MM VI, two were MM VII, and one was MM VIII (M-7.1). The building is notable for several reasons (Fig. 2): while the lower four stories of the building are conventional concrete flat slab construction the upper seventeen stories are composed of three 70'-0" bays spanning between steel frames. Except at the periphery of the floor the 70'-0" span is carried by 37" deep precast, prestressed simply supported concrete beams 10'-0" on center, each weighing 28,000# and containing holes in the web to permit the passage of air ducts, plumbing, etc. By threading mechanical equipment through the deep beams rather than under them, no increase in story height was needed despite the long spans. One half of the plan area of the center bay

* Project Engineer - Reid Rockwell Banwell & Tarics - Architects & Engineers

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is occupied by a utility core. In the walls of the utility core are located concrete encased steel braced frames which extend to the foundation and which carry the major part of all lateral loads. A small part of the lateral loads is carried by the moment resisting steel frames on which the concrete beams rest. The framing as described results in a rentable area completely free of interior columns. All interior walls outside the utility core are prefabricated light metal partitions, designed for ease of rearrangement, (a highly desirable feature in office buildings) but completely lacking the structural strength and rigidity of traditional masonry block partitions. The building is sheathed by a glass curtain wall held by aluminum mullions, an earthquake hazard unless properly detailed.

Because of the large spans and movable partitions the following structural problems exist:

(1) There are fewer load resisting and energy dissipating elements than in conventional construction.

(2) The damping of such a structure will be lower than for a traditional structure, resulting in higher stresses and larger deflections in an earthquake.

(3) The lateral stiffness of the building might be insufficient for wind and seismic loads.

Not to be overlooked however, is the structural simplicity and symmetry of the design, (a consequence of the architectural philosophy of the designers of the building) which makes it possible to apply a rather exact structural analysis, too often a practical impossibility with set back and unsymmetrical buildings. This simplicity has important and rather obvious economic and aesthetic benefits as well.

PRELIMINARY DESIGN

A preliminary design was first prepared on the basis of a steel braced cantilever tower designed for a total base shear of $3\frac{1}{2}\%$ at code stresses with 33% stress increase. In addition, moment resistant frames were placed to support the prestressed beams at the short end-walls and to help in resisting torsional oscillations. (See Fig. 2). Then the spring constants were calculated for the frames by usual methods, and the periods of vibration for the various modes were obtained using Rayleigh's method.

At this stage, it was necessary to decide what methods to use to analyze the proposed structure for the effects of a strong-motion earthquake. As a preliminary step, it was decided to calculate the approximate response of the building by the use of the well known "spectrum response theory". Curves have been published by Professors Hudson and Housner of the California Institute of Technology for many recorded earthquakes in the U.S.A. which give the maximum velocities developed by a single mass, damped vibrator as a function of the period of vibration and the damping. Such a curve, called the velocity spectrum, for the El Centro Earthquake of May 7, 1940 is shown in Fig. 3. The curves can be used to predict the response of a multi-story building if one makes the assumption that it vibrates in the first mode. Actually, of course, other modes as well are excited, but if it is assumed that all of the energy of vibration is in the first mode, one obtains a reasonable estimate of the base shear, overturning moment, and kinetic energy of vibration. Referring to Fig 3 and noting that the calculated period of the building is 2.7 sec. it is found that the maximum response velocity is about 2.2 ft/sec@ 3% damping. The corresponding base shear is then equal to $\Sigma H = 5500^{K}$. It is seen that this value is several times as large as

that specified by the code ($\Sigma H = 1600^{K}$). How this affected the design of the structure will be explained later. First it was deemed necessary to check these appropriate velocity spectrum calculations by a more exact dynamic analysis.

There are several well known mathematical methods suitable for making such an analysis, all of them depending on the integration of the differential equation of motion of a multi-mass elastic system with variable boundary conditions by one means or another, i.e.: Numerical (usually performed by a digital computer) graphical (such as the Delta-Phase Plane Method) mechanical (dynamic models) or analogous systems (analog computer).

Theoretically, all of these methods can yield accurate results. From a practical point of view, the choice of methods depends on the design being considered, the results desired, as well as on the time and cost involved. For this building Professor Housner of the California Institute of Technology was retained as consultant to perform the analysis on an analogue computer, which can be instrumented to represent a multi-story elastic system with any mass-stiffness and damping factors desired. The results of this analysis showed that the simple velocity spectrum calculations were somewhat too low, and that the expected shears and accelerations would be those corresponding to 15%g base shear (31% g at roof, 0 at ground level), $\Sigma H = 7150$. Similar forces were found to act on the building from the 1949 Olympia Washington ground motion ($\Sigma H = 6250^{K}$). These accelerations. shears and displacements are all much greater (by a factor of 4), than the code design values, and to design the structure to resist them without exceeding the elastic limit is very difficult and costly. This can be shown as follows: Assuming that all lateral load in the N-S direction is resisted by two K braced frames, 35' wide, 230' high, placed within the core walls: then for the El Centro earthquake, if the N-S base shear is 7150^{K}, the load on each truss is 3575^{K} and the force in each diagonal 2500^{K}. The code load for which the diagonal should be designed is 630^{K} and the area required is 22.5 in^2. The El Centro stress in the member is 110 ksi, which is much greater than the yield strength of the steel and the member has been stretched plastically an unknown amount, or has buckled. If a member is wanted which will just start yielding, its area is $\frac{2500}{33} = 76\ in^2$; this would be provided by a 14WF237 column section.

This last solution leaves much to be desired, however, in the way of economy as well as structurally because:

1. It is obviously uneconomical - requiring steel areas considerably in excess of those required by the code.

2. A member of this weight and compact section is very difficult to connect with rivets or bolts in such a way as to develop its yield strength.

3. Welding of the thick steel plates is difficult to do in the field, and is accompanied by uncertainties regarding the ultimate ductile behavior of an overstressed joint essential to the structural safety of the whole building.

A better approach would be to calculate the amount of plastic yielding and corresponding permanent set which would enable the structure to dissipate sufficient energy to remain stable. Many structures have stood up for this very reason - watertanks, refinery vessels, and even buildings. In many of these structures post-earthquake examination revealed that some plastic deformation had taken place, sometimes accompanied by failure of local character. In water tanks and refinery towers tie rods and anchor bolts have been stretched.

Several multi-story steel buildings in Japan and Mexico City have been observed to remain standing after quakes with horizontal dislocations of over an inch at some floor levels, indicating plastic bending and permanent set in the steel frame.

Today (1960) two difficulties stand in the way of a building designer trying to apply plastic energy absorption principles to a multi-story design - one is theoretical, the other one practical.

The theoretical problem is of great importance, and may be stated thusly; given a particular ground motion, and a proposed frame structure, what will be the location, sequence (both in time and space) and magnitude of the plastic deformations? The answer to this problem can be obtained, at least theoretically, using well known laws of mechanics. The computational work involved, however, makes the solution feasible only on a computer. Several research programs for the analysis of elasto-plastic frames are now finally being developed, but at the time of the design they were unavailable. This research activity could be of great benefit to the art of aseismic building design. It is to be hoped that methods of analysis will evolve from this work which will be as useful in predicting the response of elasto-plastic systems as the velocity spectrum is in predicting the response of elastic systems.

However, even assuming elasto-plastic behavior can be calculated, the practical problem cannot be avoided: What may be considered a tolerable deformation of the structure? This obviously will depend on the use and location of the building and has no sharply defined limit; implicitly or explicitly, in his design, the engineer solves this problem by judgment. Since the root of this problem lies in economics and psychology, it is very doubtful simple answers can be found.

The following example from the Norton Building can illustrate the difficulties involved in allowing the steel frame to yield; The preliminary design assumed a heavy moment resistant steel frame of built up members could be designed to take all seismic forces. It would coincide in plan with the walls around the core. Since the vertical load on a typical corner column in the core is 11,500k (dead load and code seismic), a heavy built up shape weighing over 2000 lb/ft is required. If this section were subjected to large bending moments, it could hardly be expected to develop its theoretical plastic moment capacity (already reduced by the vertical load) because there simply is not enough room for the required high strength bolts or rivets which carry the shear between the plates. However, assuming that the columns would not fail in shear, and that they all could develop their theoretical plastic moments M'_p simultaneously irrespective of some differences in relative rigidities, it is possible to calculate the effect of plastic hinges on the dissipation of energy in the structure. $\Sigma M'_p$ for the twelve moment-connected columns is $= 620{,}000^{K\text{-}IN}$. The base shear ΣH_{ULT} corresponding to this is $\frac{2 \cdot \Sigma M'_p}{h} = \frac{2 \cdot 620{,}000}{120^{IN}} = 5200^{K}$ (It is to be noted that this is considerably less than the maximum El Centro shear predicted for an infinitely elastic system with 3% damping.) The energy dissipated by the plastic hinges when a permanent dislocation or set of 1" occurs in the frame is $\Sigma H_{ULT} \cdot \Delta_p = 5200 \cdot 1^{K\text{-}IN} = 435^{K\text{-}FT}$ If it is assumed that such hinges occur twice in each cycle of vibration, then the average energy absorbed per second is equal to $e = \frac{435 \cdot 2}{2.7} = 324^{K\text{-}FT/SEC}$

This should be compared with the energy dissipated by 3% of critical damping, which for this structure and the calculated El Centro amplitudes of vibration* is found to be 1300k-ft/cycle, or an average of 485k-ft/sec. It is seen that the energy absorbed by hinges resulting in 1" sets is rather small, and therefore, the plastic sets would have to be quite large to give some effective damping.

* See sample calculation, page 12.

(The upper limit on the maximum possible single plastic set can be calculated approximately as follows: The maximum kinetic energy of the infinitely elastic structure at any time is $E_{max} = \frac{1}{2} M S_v^2 = \frac{2700 \cdot 17 \cdot 2.2^2}{2 \cdot 32.2} = 3450^{K\text{-}FT}$. If all of this energy were suddenly dissipated by plastic hinges between the 2nd and 3rd floor, the resulting set would be $\Delta_p = E_{max} / \Sigma H_{ULT} = 3450/5200 = 0.67^{FT} = 8$ in. (Even much less than half of this set would probably render the structure dangerously useless.) In view of the questionable assumptions regarding the plastic capacities of the columns, andthe lack of a thorough elasto-plastic analysis to establish the magnitudes and locations of the probable permanent sets, it was decided to relieve the steel frame of all substantial moments by encasing it in an expendable concrete shearwall. Other problems also appeared to be solved by the introduction of concrete.

1. The concrete would crack and absorb energy before the steel frame would deform sufficiently to yield, and therefore damping would be increased.

2. The lateral stiffness would be improved, resulting in smaller deflections from windstorms and earthquakes.

3. The use of concrete would provide lateral stability to the columns, some of which carry very heavy axial loads during the El Centro earthquake, as the following table shows:

Loads on corner core column.	
D + L = 7460^k	f_a = 13.6 ksi
D + L + Code Seismic = 11,560^k	f_a = 21.5 ksi
D only + El Centro Seismic = 23,000^k	f_a = 44.0 ksi

A steel frame was still retained, both to serve as a construction aid and more importantly to provide ductility and better ultimate strength to the structure. It was designed for a nominal value of 2.3% g base shear at code stresses. Fig. 5 shows the connections required for these forces alone.

One of the effects of the concrete is to increase the rigidity of the structure considerably. It then was necessary to consider what effect this would have on the response of the structure. The period of vibration in the first mode was found to be 1.6 sec. Referring again to the spectrum curve for El Centro it is seen that the base shear corresponding to this is 5350^k, i.e., there is no radical change in the base shear due to the shortening of the period. The shear walls were designed to carry these shears, while the steel columns would take the overturning moment.

The energy approach also throws some light on the nature of the overturning problem. It is only necessary to calculate the maximum kinetic energy available for overturning the building; this is obtained from the maximum velocity:

$E_{max} = \frac{1}{2} M S_v^2 = \frac{1}{2} \cdot \frac{3300 \cdot 17 \cdot 1.0^2}{32.2} = 870^{K\text{-}FT}$

This amount of energy is equivalent to raising the center of gravity of the building $870/3300 \cdot 17 \cdot 12 = 0.16^{IN}$. Obviously, unless the overturning can occur by failure of the compression columns or by a frame or shear failure, the overturning stability of most buildings could never be seriously endangered, even if full anchorage of the tension columns is not provided. Clearly, overturning is much more of a problem in light, slender structures such as stacks, chimneys and refinery towers, than in buildings.

Together the concrete and steel would be able to carry El Centro forces, the steel at stresses near yield point and the concrete at average stresses of about 500 psi in shear or post cracking. The concrete is well tied into the steel frame

frame, however, with shear connections and welded reinforcing bars, as Fig. 8 shows. It is expected that the structure will survive such a ground motion without substantial damage except some cracking of the concrete.

The shears and overturning moments from the tower are resisted by the very rigid and strong box shaped four story garage structure, which distributes these loads to the foundations, consisting of large spread footings. The core rests on a reinforced concrete mat approximately 80' x 80' and 7' thick (Fig. 9). The subgrade is fairly dense sand. The effect of subgrade compliance was found to be negligible in reducing the vibratory response of the building, the main effect being a lengthening of the period of vibration by about 5%.

DESIGN NO. II: PROPOSED HEALTH SCIENCES INSTRUCTION AND RESEARCH BUILDING FOR UNIVERSITY OF CALIFORNIA MEDICAL CENTER
Architects and Engineers: Reid, Rockwell, Banwell & Tarics

The proposed building is a 16* story steel-frame structure resting on 16 columns located at the periphery of the main unobstructed floor space which measured 90' x 90' (Fig. 10). This span is bridged by a gridwork of continuous 42" deep welded steel girders which are fixed at their supports by the columns, resulting in an effective space frame. The unobstructed floor spaces and the special mechanical systems lend themselves easily to frequent rearrangement of the laboratories, a constant problem in the rapidly changing field of medical research. The very extensive mechanical system (which can take care of bacteria, poisonous fumes and radioactive waste) requires more space than is usual in an office building, and to accomodate it efficiently, the deep floor girders have large holes in their webs at regular intervals. Because of relative rigidities, the floor gridwork participates only slightly in the resistance to lateral forces which is mainly provided by the spandrel frames on the periphery of the building. The architectural construction of the building is such that it does not add any elements of stiffness to the frame. (Fireproofing is of plaster or asbestos compounds, partitions are of the prefabricated light metal, self-braced type, and all stairs elevators, etc. are in a rigid tower separate from the main building). The frame rests partially on bedrock and partially on a very rigid concrete frame which goes down to bedrock. Because of the greater than usual forces on the main structural elements, all columns, girders and spandrels are fabricated from steel plates and some rolled shapes.

From economy in fabrication all columns and beams maintain their width and depth constant from top to bottom, although plate thicknesses change. Because of the summetry of the building, there are only two column and two girder types. The resultant savings in fabrication costs are expected to help offset the higher tonnage of steel required to span 93'. Members not participating in seismic resistance are connected with high-strength bearing bolts to reduce joint slippage, but seismic members are connected with ordinary high-strength bolts to permit slippage under seismic action. (This allows substantial energy dissipation to take place before any considerable yielding occurs). The aseismic design was based on a preliminary study of a model designed especially for the project.

PRELIMINARY SEISMIC DESIGN

The building is essentially a uniform mass, variable stiffness shear beam. According to the San Francisco Code, it should be designed for a total base shear of 3.5 g which in this case is about 800^{k}. The usual 33% increase in allowable stresses is permitted for aseismic design.

* later changed to 15 stories for architectural reasons.

It may be noted here that in computing the stiffness factors of the frame, it was found necessary to include the effect of shear deformation of the members. Because of the rather short and deep steel members, it was found that shear deformation reduced the effective stiffness of a joint by as much as 30% in some cases. To facilitate the analysis and design of the optimum frame, Mr. Ted York, a consulting engineer of San Francisco, was commissioned to prepare a computer program which would analyze a given frame for a given set of forces. The program assumes elastic action of the frame, but otherwise is completely general. (A 50 story frame 10 bays wide can be handled). It includes in the analysis the effect of axial column deformations on the stress distribution, and prints out all moments and forces acting on every member. In addition, it computes the deflected shape of the frame as well as the period of vibration associated with the deflected shape. The relegation of this numerical work to the computer permits the engineer to compare more alternative frame designs than would be feasible if some approximate but nevertheless slower method of analysis were used.

The period of vibration in the 1st mode was found to be 2.2 sec. According to the velocity spectrum theory, the approximate maximum base shear for the El Centro earthquake should be: 2740k, or approximately 13.0% g; assuming 3% damping, the maximum energy attained by the structure is 1290k-ft (this is equivalent to lifting the building.75"off the ground).

Since there are two frames resisting these loads, this means 1370k/FRAME An analysis of the frames revealed that they would be able to resist these loads without plastic sets, but that certain portions of the crossection of some members would be very close to yielding. By proper design of the various elements it is possible to control the location of the initial yielding and to confine it. It is also important to insure that the elements which will yield first are able to absorb substantial amounts of energy without fracturing and that the frame will not be susceptible to elastic or plastic instability; therefore, a stability analysis was made for all important members, for both elastic and plastic conditions. Column to spandrel connections were designed to transmit the forces of El Centro at approximate yield stresses. Fig. 13 shows a typical frame detail near the base of the building.

Adjunct to the problem of stress in the frame is the question of deflection. The question of how to determine the right proportions of members of the frame involves several interesting problems of importance to the ultimate safety of the building. The most important member is of course the column. This is so because the column, in addition to carrying the lateral moment also carries large vertical load. These loads would contribute, in turn, to the moment if any large deformation occured due to the moments. The spandrels, by contrast, are carrying substantially no loads except the lateral, and they could accept moderate yielding. In many tall frame buildings, the factor which governs the sizes of the members in the lateral frame is not the lateral stress so much as the permissible drift (or horizontal deflection of the roof) due to lateral load from wind or earthquake. The practice of limiting the calculated deflection of the steel frame to two one thousands of the height under the code wind load is based on extensive experience with tall structures in the U.S.A. which indicates that buildings so designed do not sway objectionably in strong windstorms. A factor to be considered, however, is that masonry elements in traditional steel frame buildings added mass and stiffness to the building both of which would tend to reduce the amplitude of gust-induced swaying; many engineers estimate that the increase in stiffness of a typical traditional building due to the masonry may be as large as by a factor of 2 or 3. Modern frame buildings, lacking masonry, rely much more on the steel

frame, which must supply the required rigidity. Fortunately, it is generally found that the columns contribute little to the overall deflection, so that for a small increase in girder size a substantial stiffening of the building is possible. In this building, enough fasteners are generally provided in the spandrel moment connection to develop the ultimate strength of the girder. This often results in a greater number of bolts than would be required for the code seismic moment especially in members having low D & L stresses. Since the increase in the safety factor is large compared with the cost of the additional bolts, the investment is felt to be worthwhile.

MODEL ANALYSIS AND DESIGN

For a more thorough dynamic investigation it was decided to build a simple model. The purpose of the model was to give a better intuitive and visual understanding of the behavior of a building in an earthquake than had been obtained by purely mathematical methods. The structural engineer depends very heavily on his visual sense of proportions to detect errors in design or to solve problems which often are not susceptible to analytical methods. It was felt that the very minor investment in a simple mechanical model would repay itself in the ability to visualize a physically simple process, the mathematics of which happens to be cumbersome. If, in addition to that, it were found that quantitative answers could be obtained with reasonable accuracy, the model would be completely justified. When designing the model, it was decided to make the horizontal deflections of the model equal to that of the prototype for identical ground motions. This was done because it is easy to relate deflections to stresses, (at least in the elastic range) and the deflections are easily measured on the model.

Since analysis of the proposed steel frame showed that 80% of the deflection is caused by bending and shear deformation of the frame, and only 20% by axial stresses causing rotation of the floors, it was clear that a very close dynamic approximation could be obtained by a "shear-type" model. The model consists of 16 plywood panels 18"x18" weighing 3.66 lb each mounted on a shaking table actuated by a groove cut to the proper scale from actual horizontal ground displacement-time curves as obtained from U.S. Coast and Geodetic Survey seismographs. The recording of the deflections is accomplished by mounting on a given floor very light smoked paper drums rotated by small electric motors at a constant speed, and having the relative motion between floors inscribed by a levered scratch arm. To duplicate the frame stiffness characteristics of the prototype, the distance between model floor masses is varied as shown in Fig. 17. In computing the stiffness factors (K_m) of model, the effect of vertical load on stiffness was taken into account. This factor, which proved to be quite troublesome had much to do with the final proportions of the model. Since it is physically very difficult to build a stable and reasonably small model with a period as long as the prototype's (2.3 sec), the model time scale was compressed 2.3 times. To retain dynamic similitude, the rate of introduction of the ground motion function had to be correspondingly increased. The dimensional relations between the model and prototype which allow the calculation of any variable in the prototype, are obtained by usual methods of dimensional analysis.

The damping of the model was studied and was found to increase with amplitude, as it undoubtedly would in the prototype. It was found that the model damping represents a prototype damping of 3% g @ deflections 1/8"/floor, 5% @ deflections $\frac{1}{4}$"/floor, 8% @ deflections $\frac{1}{2}$"/floor, 10% @ deflections 1"/floor.

The first question answered by the model had to do with the behavior of the

lower floors during an earthquake. Although it was generally believed that the maximum relative displacements between stories do not occur at the beginning of an earthquake but rather build up as energy is accumulated in the building, it was nevertheless questioned whether some of the first large ground displacements would not be able to yield the lower floors. An examination of Fig. 14 showing the relative displacement between the ground floor and the 2nd floor shows that in the El Centro 1940 earthquake, at least, this does not occur. The maximum displacements and base shears are reached about 8 seconds after the start of ground motion. At that time a condition seems to be reached by the structure which for lack of a better term might be called a semi-steady state of forced vibration, since the maximum base shear periodically repeats itself, each time reaching approximately the same maximum value. This is of interest, since if the base shear is greater than the elastic limit shear of the frame, it seems possible for the frame to undergo a considerable number of reversals of plastic yielding. Fig. 14 also shows that at the beginning of the ground motion the base shear is quite irregular, but as energy begins to accumulate in the building the base shear starts conforming more or less with the first mode of vibration. A record of the 12th floor displacement shows that the shear at that level is affected relatively more by higher modes, as is expected. Especially predominant is the second mode. As at the ground floor, the maximum shear is repreated several times with little variation in magnitude. The measured deflection at the 2nd floor corresponds to a base shear of 5.3 lb in the model. The corresponding base shear in the prototype is 2800^{K}. This checks well with the base shear calculated using the velocity-spectrum theory: for T = 2.3 sec. $\Sigma H = 2740$. The maximum sidesway of the model during this earthquake was obtained by simultaneous recording at the top and bottom stories, and is seen from Fig. 14 to be about 2" which corresponds to 16" on the prototype (32" double amplitude) or about 1"/floor.

Fig. 15 shows the vibration records obtained on the model for the San Francisco earthquake of 1957. The smaller amplitude and higher frequency vibrations of the smaller earthquake are seen to excite the higher modes more than the El Centro earthquake did. The 2nd floor displacement still has the 1st mode evident, but strongly modified by 2nd, 3rd and higher modes. In the 1st floor displacement curve, it is very hard to see the 1st mode at all. The computed maximum shear at the base is 340^{K} or about 1.6% g.

CONCLUSIONS

While both the Norton and Medical Science Buildings have much in common, (they are both modern, glass curtain wall enclosed steel frame buildings with interior space uncluttered by masonry partitions), it can be seen that they achieve their required lateral seismic strength by different methods which are consistent with their respective framing patterns.

The Norton Building is framed essentially by simple beams resting on long span flexible steel frames. This means that the elements which carry vertical loads cannot be used to resist seismic load, and therefore a separate structural system for these loads is necessary. This separate system was severely restricted in size by the architectural requirement that the rentable area be clear of all columns. It was necessary, therefore, to use a lateral bracing system which in many ways is comparable to the lateral bracing system of more conventional type masonry filled steel framed structures. That is, a steel frame to carry the large stresses in major earthquakes and to tie together a masonry filler wall; and a filler wall which would provide the stiffness for small earthquakes and wind loads and provide damping and energy absorbtion for large earthquakes. It

is true, the space requirement necessitated condensing this system, as it were, to only the core area, but in essence it remains comparable to the conventional masonry fill steel frame building.

The Medical Science Building by contrast has no core area within the building to be used for lateral strength. This requires a rather substantial departure from conventional design, that is, a frame which will resist adequately all lateral forces with minimum deflection and adequate ultimate strength. This requirement was not difficult to achieve, however, thanks to the way the vertical loads are carried; that is, with large span steel girders which are fixed to the columns, resulting in large column moments and therefore large column sections, and relatively short distances between the columns, due to the fact that all columns are near the outside of the building. To a scheme designed primarily to carry vertical loads, it was then only necessary to add a relatively small amount of steel in the form of spandrel beams and connections to achieve an extremely rigid and strong 3 bay frame, which could carry all conceivable lateral forces. The inherent strength of this frame can be shown by comparing the ultimate base shear capacity of the steel columns to the weight of the building, for both building (see page 12.) The Medical Science Building has approximately twice the base shear capacity of weight ration as the Norton Building. This difference is balanced of course by the strength and damping capacity of the concrete shear wall .

The concept and therefore the solution to the seismic strength of both buildings is different but, from a very basic point of view, the problem is the same. This problem, which is sure to confront the engineer more often in the future is; what criteria can be used for the seismic design of structures which do not conform to conventional building practices. In these cases codes and experience alone fail and extrapolation of past experience to unusual conditions must be accompanied by considerable intuition and judgment. Surely the criteria of tomorrow will in some degree be written by the failures of today. The seismologist and mathematician can help the engineer by developing more workable theories and in recent years much of value has been done by them. It must be remembered however that from the engineer's point of view any usable method of seismic analysis must be simple to apply. Many engineering decisions which involve considerable cost to the client and the safety of the public must be made in a relatively short time and the responsibility for these decisions rest heavily and squarely upon the engineer, as they should. It is with gratitude therefore that the engineer accepts the tools which have been given him.

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NOMENCLATURE

ϵ = average rate of energy dissipation

$\bar{E}$ = maximum energy in the building

F = force per unit height of building which causes deflection

h = height of building

ΣH = total base shear

KE = kinetic energy

m = mass per unit height of building

S_v = velocity response, maximum velocity attained by single mass system acted on by given force function as a function of damping and period.

T = period of vibration

x = distance measured vertically on building

Y = horizontal displacement of building due to imposed force, or shape of first mode.

Y = maximum horizontal displacement of building due to given force function

$\dot{Y}, \ddot{Y}$ = 1st and 2nd derivative of Y, maximum velocity and maximum acceleration.

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SAMPLE CALCULATIONS FOR THE MEDICAL SCIENCE BUILDING

Properties of the building:

Height - 16 stories	210 feet
Weight - 1300 kips/story	90 k/ft

A uniformly varying load of 0 @ the base to 100 k/str. @ the roof gives a deflection of 3.8" @ the roof.

Period: $PE = \frac{1}{2}\int_0^h F\cdot y\cdot dx = \frac{1}{2}\int_0^{16} \frac{100}{16}x \cdot \frac{3.8}{16}x\,dx$

$KE = \frac{1}{2}\int_0^h m(\dot{y})^2 dx = \frac{1}{2}\int_0^{16} \frac{1300}{g}\left(p\,\frac{3.8}{16}x\right)^2 dx$

$PE = KE \quad p^2 = \frac{100}{1300}\frac{g}{3.8}$; $T = 2\pi\cdot\frac{1}{p} = \frac{2\pi\cdot 1}{2.83} = 2.22$ sec

The velocity response for El Centro @ 3% damping

For a single mass system $S_v = 2.0$ FT/SEC

For a multi-mass system in the 1st mode

$$\dot{Y} = \frac{\int_0^h my\,dx}{\int_0^h my^2 dx}\, y\, S_v$$

Assuming the shape of the 1st mode is a straight line

$$\dot{Y} = \frac{\int_0^{16} \frac{1300}{g}x\,dx}{\int_0^{16} \frac{1300}{g}\frac{x^2}{16}dx}\cdot\frac{x}{16}S_v = \frac{3}{2}\frac{x}{16}S_v$$

@ the roof

$\dot{Y} = 1.5\,S_v = 3.0$ FT/SEC

$Y = \dot{Y}/p = 3.0(12)/2.83 = 12.7"$ @ top

$\ddot{Y} = pY = 3.0\cdot 2.83 = 8.5$ FT/SEC^2 $= 26.4\,\%g$

base shear

$\Sigma H = \frac{1}{2}\,16\cdot 1300\cdot 0.264 = 2740^k$

Maximum energy in the building

$\bar{E} = \frac{1}{2}mh(S_v)^2 = \frac{1}{2}\frac{1300\cdot 16}{32.2}(2.0)^2 = 1290^{K\text{-}FT}$

Maximum average energy dissipation

$e = \frac{4\pi}{T}(\%\text{ damping})\cdot\bar{E} = \frac{4\pi}{2.22}(0.03)1290 = 218^{KFT/S} = 400^{HP}$

COMPARISON OF ULTIMATE BASE SHEAR TO TOTAL WEIGHT RATIOS FOR THE TWO BUILDINGS

Norton Building, Preliminary Design Steel Frame Only: $\frac{\Sigma H_{ult}}{W} = \frac{5200}{46{,}000} = 0.113$

Medical Science Building, Steel Frame: $\frac{\Sigma H_{ult}}{W} = \frac{5550}{21{,}000} = 0.265$

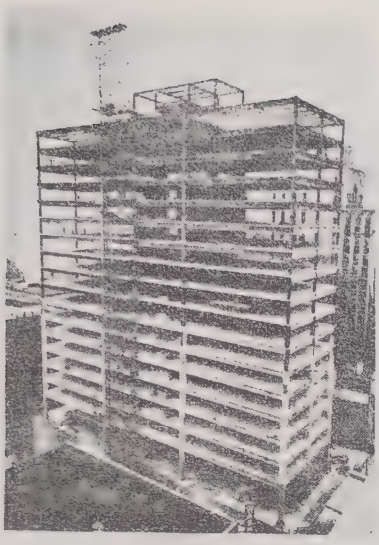

Fig. 1. Perspective of Norton Building under construction.

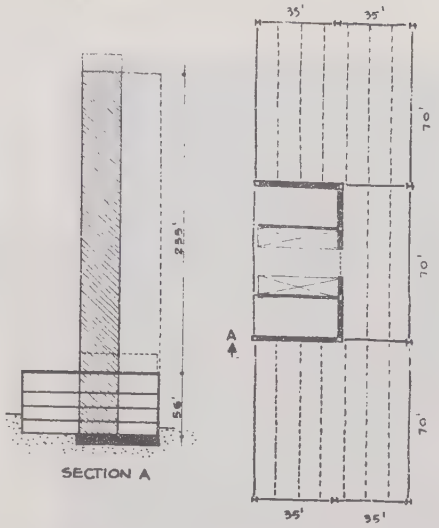

Fig. 2. Plan and Section of Norton Building

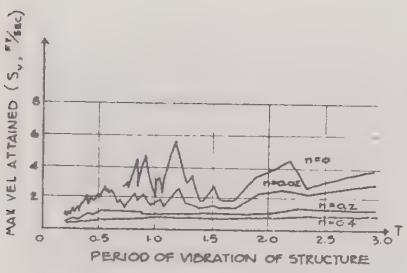

Fig. 3.

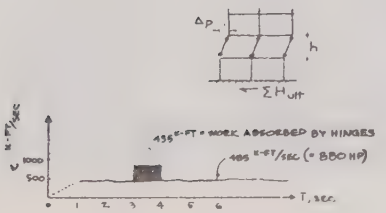

Fig. 4.

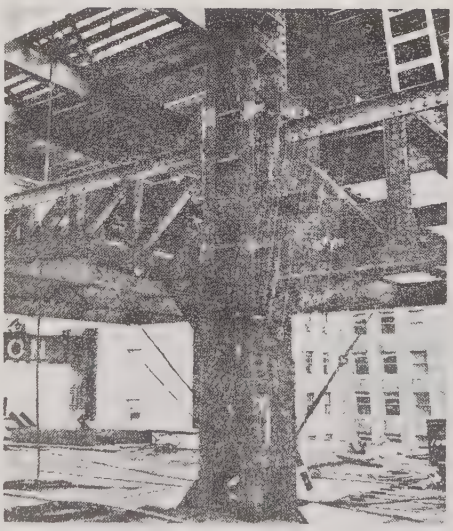

Fig. 5. Steel Frame detail on 5th floor

Fig. 6. View of Core framing - steel only

Fig. 7. View of frame going up

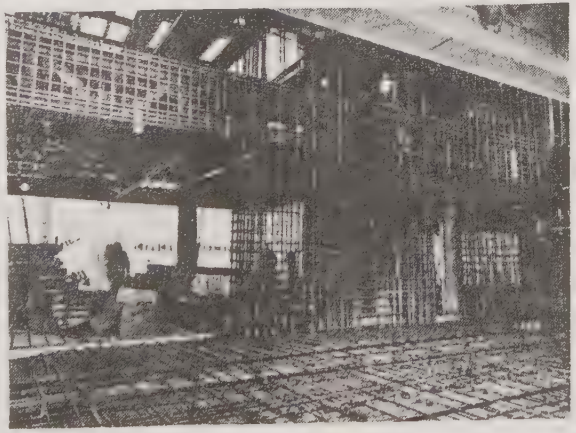

Fig. 8. View of core framing - reiforcing in place

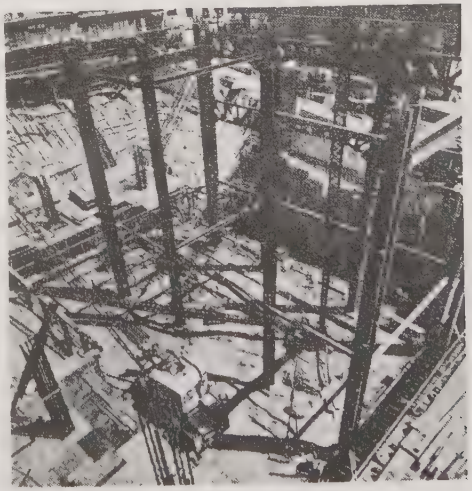

Fig. 9 View of core columns framing into foundation.

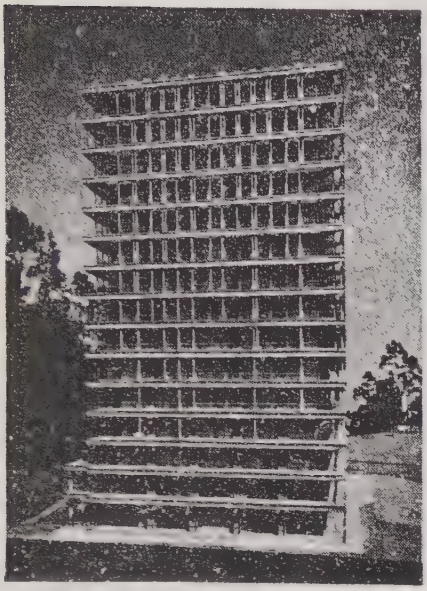

Fig. 10 Model of Health Science Research Building

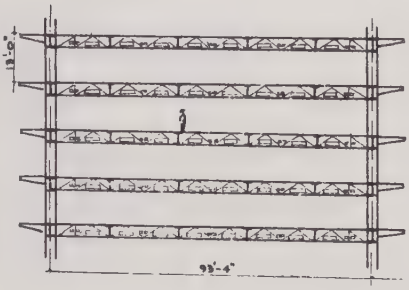

Fig. 12 Section through floor

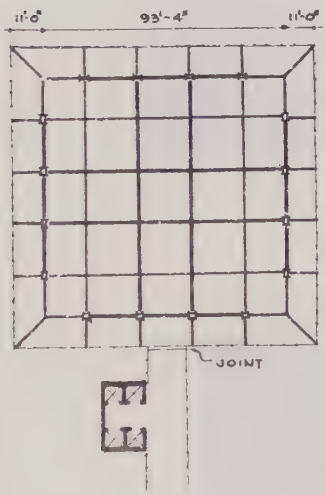

Fig. 11 Floor Plan

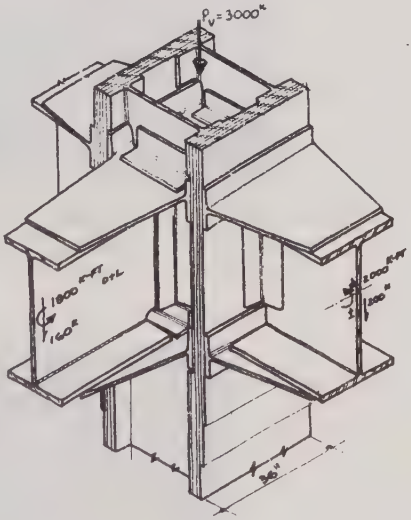

Fig. 13 Typical column and girder connection

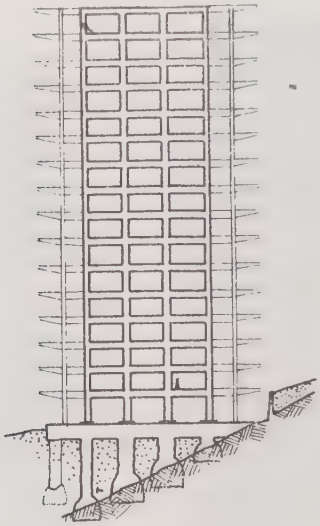

Fig. 16 Elevation of frame

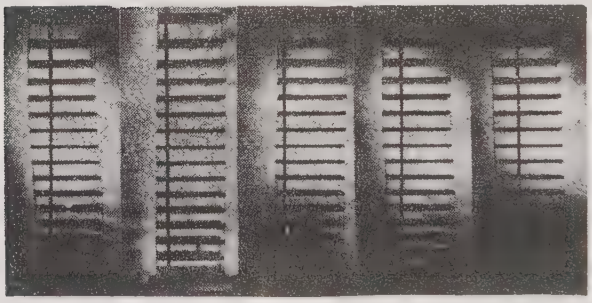

FIG 17. DEFORMATION OF MODEL DURING SHAKING TABLE TEST - EL CENTRO 1940 EQ

Fig. 17

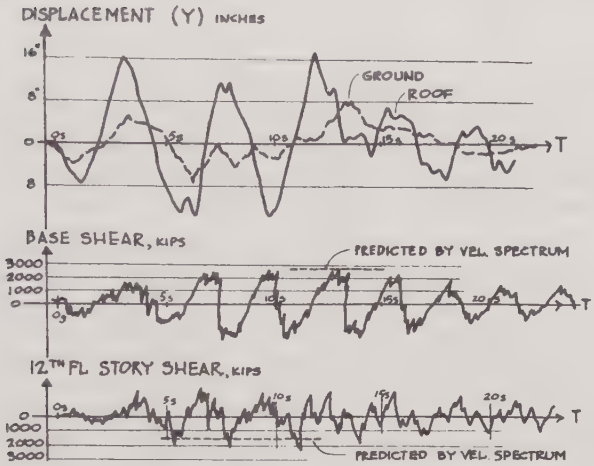

FIG 14: RESPONSE TO EL CENTRO 1940 GROUND MOTION

Fig.

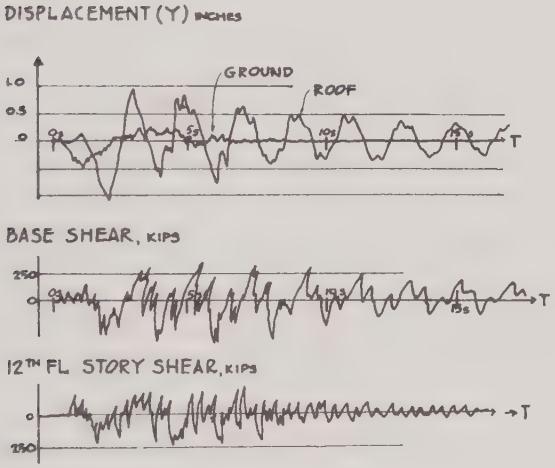

FIG 15: RESPONSE TO S.F 1957 GROUND MOTION

Fig. 15

DESIGN OF EARTHQUAKE - PROOF BUILDING STRUCTURES IN THE USSR

By

V.A. Bikhovsky*, J.I. Goldenblat* and J.L. Korchynski*

INTRODUCTION

While establishing seismic regions on the territory of the USSR, it was found out that in the country there are about 5,5 sq.km. of land where earthquakes whose intensity is equal to 6 - 9 Mercalli-Sieberg seismic scale. Some 50 million people inhabit this territory. Naturally, the huge size of the capital construction according to the 7-year plan of the development of the USSR national ecomony makes exceedingly important the problems of prviding stability of buildings and structures during possible earthquakes. For eleven Union Republics (of fifteen Union Republics forming the USSR) problems of antiseismicity are problems of state importance. These problems are regulated by special All-Union Standards and Codes for building, having obligatory force for seismic regions on the territory of these Union Republics. The total sum of 3 trillion allocated for building according to the seven-year plan of 1951 - 1965 is distributed so that 25 percent of the capital investments are spared for building in seismic regions of the Union Republics. Therefore building economy is one of significant factors in seismic regions. The main problem is to lower additional costs for antiseismic measures, caused by the necessity of life protection during earthquakes and preservation of their labour results by means of building antiseismic structures capable to with stand earthquakes without destruction. Investigations of antiseismicity of buildings and structures in the USSR are carried on in 7 scientific research institutes located in seismic regions (Georgia, Amrmenia, Uzbekistan, Turkmenia, Azerbaidjan, Kazakhstan and Tadjikistan). Theoretical investigations of antiseismicity are carried on in Central Scientific Research Institute of Building Structures (Building and Architecture Academy of the USSR). This paper presents a survey of investigations carried on at the institute.

STRONG MOTION EARTHQUAKES FOR THE PERIOD OF 1902 - 1959

14 strong motion earthquakes were registered on the territory of the Soviet Union in the 20th century; some of them caused death of people and destroying a number of buildings (see the Table).

Nos.	The Name of An Earthquake	Date	Note
1	2	3	4
1	Shemakhinskoye	1902	Many buildings were destroyed and damaged

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2	Andijanskoye	1902	Many buildings were destroyed and damaged. There were victims
3	Alma-Atinskoye (Vernenskoye)	1911	"
4	Leninakanskoye	1926	"
5	Crimean	1927	Many building were destroyed and damaged
6	Zangezurskoye	1931	"
7	Karpatskoye	1940	"
8	Chatkalskoye	1946	"
9	Ashkhabadskoye	1948	A great many buildings were destroyed, many people perished
10	Khaitskoye	1949	"
11	Mondinskoye	1951	Some buildings were damaged.
12	Kurilskoye	1952	Tsunami. Many buildings were washed away by waves. There were victims
13	Muiskoye	1959	Some buildings were damaged
14	Kamchatskoye	1959	Many buildings were damaged. Some buildings were destroyed. There were victims.

Thus, for the past period 1 strong motion earthquake occured every 4-5 years in the USSR. It can easily be compared with well-known periodicity of strong motion earthquakes in California (USA) and Japan.

State Standartization of Building in Seismic Regions of the USSR

Up to 1940 building in seismic regions of the USSR was regulated in some areas (Kazakhstan, Crimea and Transcaucasus) by Specifications published by local authorities which had not All-Union recognition. In 1940 for the first time was adopted "Instruction for designing of Civil and Industrial Buildings and Constructions erected in Seismic

regions," which was used all over the USSR.

In 1943 it was somewhat revised for the conditions of war time and was used in this edition up to 1948, when it was replaced by more detailed "Specifications for designing of buildings and structures in seismic regions" (TY-58-48). In 1951 were published "Regulations concerning building in seismic regions" ПСП-101-51),[2] which were the first All-Union Normative Document obligatory for use in building in all seismic regions of the USSR. Finally, in 1957 were adopted new "Standards and Codes for Building in Seismic Regions" (CH-8-57)[1] which essentially differ from all former normative documents.

In this normative document there are nine sections and the appendixes. In the first section are given instructions concerning the seismicity of the region and design seismicity of buildings and structures. In the second section are given instructions concerning the planning of towns and villages. In the third section are given the most important new instructions concerning seismic loads for dwelling, civil, industrial and agricultural buildings and structures. In the fourth section are given antiseismic measures for industrial and civil buildings and structures, the majority of instructions being concerned with the provision of antiseismicity of stone buildings. In the fifth section are given instructions concerning the building of water-and sewer pipes. In the sixth and seventh sections are given instructions for the design and antiseismic measures for road and hydrotechnical structures. In the eighth section are given instructions concerning the building in rural areas. In the nineth section are given requirements concerning carrying out works and control for carrying out antiseismic measures.

In the appendixes are given maps of seismic regions and enumerated some places, seismicity being pointed out for them. We are going to state here not all the contents of the standard CH-8-58 but only considerations and grounds which refer to a new dynamic method of determining design seismic loads for buildings and structures. This method essentially differs from the so-called "static" method of design which is used up to the present time in many countries including Japan where it was worked out and proposed by F. Omori and K. Sano, well-known Japanese scientists, in the beginning of the present centure for practical use.

Practice introduced amendments into this method of design (which was used in the USSR up to 1957) by bringing in the correction factor $\mathcal{L}$[2] The design value of seismic force was determined by means of a formula

$$S = \mathcal{L} K_{\mathcal{L}} Q$$

where $\mathcal{L}$ - a certain correction factor;

K_c - seismic coefficient;

Q - the weight of an element of the construction located on a certain level of the building or structure.

Inadequacy of such approach to the design of structures for seismic forces is universally recognized. In the USSR the first suggestion concerning the transition from the "static" method of design under the action of a shifting base was made by K. S. Zavriev and A. G. Nazarov in 1931.(3) They proposed to proceed from the premise that the earth motion during an earthquake may be liken to harmonic forced (beginning in accordance with the cosine law) oscillations. The discussion on this problem which began then did not lead to a practical result and the method of design remained as before based on "static theory".

Beginning with 1948 the Central Scientific Research Institute of Building Structures (Building and Architecture Academy of the USSR), previously ZNIPS, commenced the work on the improvement of the design method of a seismic load.

The consideration of the elastic system oscillations with "n" number of degrees of freedom under the action of a horizontally shifting base (Fig. 1) shows(4) that forces appearing in such system are equivalent to forces S_j, which later we shall call seismic forces and which can be expressed in the following way:

$$S_K = \sum_1^n S_{Ki} = \sum_1^n K_c \beta_{it} \eta_{Ki} Q_K$$

where S_{ki} - force at the point k with the deflection of the system in i - main direction;

K_c - coefficient, determining the relation between the acceleration of a shifting base and the acceleration of gravity (g), i.e. the value determining the seismic intensity of the region;

β_{it} - coefficient, determining the dynamic effect of the displacement base action to the system in consideration in each i - main direction at each currant moment of time t;

η_{ki} - coefficient taking into account the influence of the deflection form of the system in a corresponding i - main direction, equal to

$$\eta_{Ki} = \frac{X_i(x_K) \sum_1^n Q_j X_i(x_j)}{\sum_1^n Q_j X_i^2(x_j)}$$

$X_i(x_k)$ and $X_i(x_j)$ - deflection of the system at its free oscillations in i - main direction at the point k and all points j, i.e. at points corresponding to the location of loads in adopted design scheme (Fig. 1) of a structure;

Q_k and Q_j - weights of the masses at the point k and all the points j of the system;

n - number of freedom degrees of the system, i.e. number of places where loads Q are located;

g - gravity acceleration;

The expression (2) shows that the value of seismic force S_k presents the sum of series of addends (S_{ki}); therefore it is natural that the design which takes into account all addends of the series is more perfect. However, at such summerizing it is necessary to remember that multiplicators β_{it} are values which change depending on time, therefore the determination of S_k as the sum of addends is very embarrassing and practically unattainable.

The examination of earthquake seismograms shows that usually every earthquake begins with oscillatory processes of high frequency, which are similar to the impulse action, and then passes to oscillations with lower and lower frequencies. The influences of such oscillatory processes upon systems with various frequency characteristics of free oscillations will be various as it is known. The dynamic effect of the impulse upon rigid systems with high frequency of self-oscillations will be essential and its action will be almost simultaneous with the moment of the impulse appearance. On the contrary, the maximum deflection in systems possessing low frequencies will correspond to the action of subsequent part of the seismogram and they will begin later, only after repeated oscillatory action of the base.

Taking this into account it should be noted that maximum values of separate addends of the series (2), i.e. forces S_{ki} reacting to oscillations of the system in accordance to its various forms of free oscillations usually possessing essentially different frequencies will also be mutually shifted by time. Seismic forces corresponding to a high frequency of the structure appear and get their maximum value at that moment when oscillations corresponding to a low frequency of the structure do not develop yet. To the moment when the latter get their maximum value the former will already be insignificant or completely damped (Fig. 2). Taking this into account it was decided that for practical purposes it was more expedient to consider separately all possible maximum values of forces in each main direction (S_{ji}) and to design the structure for the most disadvantageous of all possible combinations.

In this case the necessity of determining the time variable β_{it} falls away and only its maximum value, which we shall write without index t i.e. presents some practical interest.

Thus, principle scheme of the design of the structure for seismic influences, or to be more precise, the scheme of design for determining seismic loads consists in calculating several groups of possible loads corresponding to the forms of free oscillations of the system and

determined by the formula:

$$S_{kl} = K_c \beta_l \eta_{kl} Q_k \qquad (3)$$

In the formula written above coefficients K_c and η_{kl} are quite definite and do not require special explanation. The coefficient β_l depends on the base motion law whose character during the earthquake is rather complicated and intricate and in every case is somewhat different.

For the determination of β value it would be rational to use the method proposed by M. Biot (9), (10), with the help of which dynamic effect of an earthquake is found out experimentally on models. However, this method required lengthy time for its realization. The use of the available in the literature data corresponding to seismic records made on the territory of America without their even if indirect critical evaluation was somewhat risky. Therefore Soviet scientists not only carried out experimental investigations on the elucidation of the dynamic effect of earthquake on various structure models (investigations performed under the leadership of A. Nazarov in the Armenian SSR (6)) but analysed a number of seismograms.

The analysis showed that for the mathematic interpretation of soil displacement during an earthquake one may adopt the law of summary action of damped harmonic curve series, i.e.

$$y_0(t) = \sum_1^n a_0 e^{-\varepsilon_0 t} \sin(\omega t + \gamma) \qquad (4)$$

where $y_0(t)$ - soil displacement

a_0 - initial amplitude of the corresponding addend of the series.

ε_0 - coefficient of soil damping;

ω - frequency of oscillations of the corresponding addends of the series;

γ - the angle, determining the moment of appearing the corresponding addend of the series

t - time.

As is known the summary action of such harmonic curves with different parameters (i.e. different initial amplitudes a_0, frequencies ω damping ε_0 appearing at various moments of time) may cause diverse changes in the soil displacement character, practically defining any law of motion. The adoption of such soil displacement law connected with sudden change of velocity speed from zero to some final value($a_0 \omega$) at the moment of appearing of the corresponding addend of the series (4) cannot be considered quite probable, though it quite well agrees with the conception of impulse sourse of generating oscillations and agrees satisfactorily with experience.

The possibility of practical usage of adopted soil displacement law for designing structures for seismic influence required the presence of the data of parameter values such as the components of equation (4). Processing a series of seismograms registered by seismic stations in various points of the USSR* showed that parameters characterizing equation (4) are placed within rather close value limits. The periods of oscillations of the earth surface (i.e. $T = \frac{2\pi}{\omega}$) corresponding to maximum accelerations were equal to 0.25 - 0.75 sec. and mean values of decrements of damping ≈ 0.10.

Thus as an initial premise for determining β - coefficient it is supposed that the motion of soil is submitted to the law of equation (4) in which the values $\frac{2\pi\xi_0}{\omega}$ (the decrement of damping) in all addends are constant while ω - values may be of any magnitude within certain limits $\omega_1 - \omega_2$. Proceeding from these premises and making some assumptions about possible properties of series (4) and considering that the intermediate meaning of the value characterizing the damping of oscillations in building constructions (decrement of damping) is approximately equal to 0.30 there were determined β - values for systems with various periods of free oscillations T. The results of calculations are presented in Fig. 3a.

While discussing the project of standards for design of structures for seismic action which are used at the present time (1), into β - diagram (Fig. 3a) some corrections were introduced and finally it took the form presented by solid line (Fig. 3b)

It is interesting to compare β - diagram obrained with the diagram of seismic coefficient c which was adopted at California Project of Standards (8) and which was based on rich experimental material of many American earthquakes. However, it is necessary to make a reservation that c - diagram mentioned above presents some value somewhat different from β - coefficient.

C - value is the relation between maximum acceleration occuring in a structure with the period of natural oscillations T and the acceleration of gravity. Consequently, c - value is to the product of coefficients $K_c\beta$ but not to β.

In Fig. 3б there are given c - values and products $K_c\beta$ corresponding to $K_0 = 1/40$. It is easy to note that values c and $K_c\beta$ although they are obtained in different ways are highly similar. Apparently such result of comparison should be considered favourable, for coincidence of results observed is the testimony of probability of approaching the value of dynamic action of earthquakes on structures to its real effect.

The analysis of design of real structures upon seismic action shows that for a great majority of buildings only the lowest mode of oscillation is of practical importance and consequently the design of such structures may be limited by the determination of only one group of forces S_{kl}.

* The seismograms which were used fitted for very small earthquakes; we could use only these seismograms because when shock were stronger the seismograms available became unfit for processing in view of large magnification and low speed of the tape motion.

The highest modes of oscillations, the second and in some cases the third, are essential mainly for flexible structures where belong chimneys, towers, masts, etc. Therefore the structures of this type should be designed for two or three combinations of seismic forces. Besides, β - value for the structure of this type is adopted 1.6 times greater taking into consideration the fact that they as a rule possess lesser damping.

A number of buildings subjected to the action of the earthquake the intensity of which was equal to 9 (seismic scale) after careful examination was subjected to control design according to the method described above. 14 brick buildings the height of which was equal to 1-4 stories, a reinforced concrete three-storied frame, a mill-structure (of compound construction), steel tower, supporting a water-tank, and a brick chimney 60 m high were subjected to such design. The results of control designs showed that in most cases ($\sim$ 85 - 90%) actual data concerning the destruction and safety of structures after an earthquake coincided with the results of the design. In particular it is necessary to point out the design of a brick chimney; in accordance with it the chimney should have destroyed in its upper part but not at its base where according to the former methods of design maximum stresses should be expected. In reality the upper part of the chimney destroyed.

Passing to the evaluation of the method described from the point of view of its practical fitness it should be pointed out that its main drawback is complexity and expenditure of much labour for calculations connected with the determination of periods and modes of free oscillations in various buildings and structures. If these two characteristics of a structure are known the further calculations are simple and do not require much labour. Therefore in the standard which is used at present and is called "Norms and Codes of building in seismic regions (CH - 8 - 57)" (1), together with basic principles of the method, there is a number of points which simplify to a considerable degree the technique of calculating in practical designs. For the determination of seismic forces in brick buildings which are used as dwellings there are given final values of products $\beta\eta$ and consequently there is no need intermediate calculations for determining periods (T) and modes ($\chi_{(x)}$) of oscillations of these structures. Further instructions are given permitting the simplification of calculations connected with the determination of forms of structure deformations, etc.

For the interpretation of standards in detail an instruction is worked out at the present time, it contains formulae and diagrams which make easier the calculations of T and $\chi_{(x)}$ for various types of civil and industrial buildings.

In Fig. 4 and 5 there are given data of the design of reinforced concrete chimney, 80m high (5). In Fig. 4 the modes of the chimney oscilations (4a), the corresponding values of seismic forces (4b) and shears (4c) and bending moments (4d) are presented. As the design is to reveal maximum forces of envelope shears and moments are drawn in Fig. 5.

On the Leading Principles of Designing Antiseismic Buildings

The main principle of designing antiseismic buildings and structures is dynamic design for seismic foces the grounds of which are given above. Indeed, this method of design makes possible to obtain more plausible scheme of real behaviour of building structures during earthquakes. Leaning upon this relatively plausible design the USSR Standards (1) contain indications concerning some general principles of designing antiseismic buildings. While designing buildings for seismic regions it is necessary to follow the principle of equal strength. The analysis of many designs of antiseismic buildings shows that this principle is often neglected the result of it being weak joints and elements which may be the cause of the destruction of the building before the moment when the rest of the elements of the structure exhaust their safety factor. Therefore the standards recommend to obtain as similar strength as it is possible in all elements which form the bearing construction of the building. Effective distribution of masses and rigiditiesis the essence of the following principle ensuring antiseismicity of a building. The importance of this principle is great as its application permits to reduce the values of seismic foces to their minimum. It is important to take into consideration plastic deformations of constructions occuring under seismic action.

Experimental investigations for some time past confirm the results obtained at our Institute as far as 1953 (11), (12), showing that plastic deformations which are being developed give possibility to increase essentially the strength of materials at momentary impulse action which is known are a highly characteristic type of load in the very beginning of the earthquake. In certain frame structures deformations at impulse seismic influences may be rather large (13). While designing reinforced concrete frames it is necessary to create favourable possiblilities in them for forming plastic deformations during momentary action of an earthquake. Practically it means the striving to use the least possible percentage of reinforcement compensating it by increasing the size of the section, etc.

In consequence of the constant widening of the usage of built-up reinforced concrete structures in seismic regions the principle of making built-up reinforced concrete structures monolithic is the matter of prime importance. In other words it signifies the demand for turning a built-up structure into a structure equivalent to monolithic reinforced concrete (14), (15). In a special report these problems are examined in detail.

Some Data Based on Experimental Investigations

a) During an earthquake in structures a marked dynamic load occurs, its action is short at repeated loadings the number of which is approximately equal to 100 - 100 cycles. At present there are rather few direct experimental data which give the possibility to judge of the material strength under not numerous repeated loadings. Experimental investigations of the steel strength at not numerous loadings and speeds close to those .

ones which are observed in structures during earthquakes.

Specimens of round form, 10 mm diameter, 82 mm length, were tested on the pulsator of unilateral action. The purpose of the investigation was the establishment of the interrelation between the strength of the material and the number of cycles of loading. In the work of one of the authors (16) a suggestion was made that the interrelation between the value of the destructive stress and the logarithm of the number of repeated loadings is linear. The experiments made confirmed for steel specimens the existance of such dependance. In Fig. 6 experimental confirmation of the hypothesis adopted is illustrated (17); along X - axis the logarithm of the number of loading cycles and along Y - axis the relation between the stress at dynamic destruction of the specimen and its limit stress at static loading were marked. Hence it is seen that at short action of a few repeated loadings the steel strength increases by 20 percent. This result is taken into consideration in Standards (1) by introducing an additional coefficient raising stresses in steel during an earthquake.

b) According to the new method for the determination of design seismic load on the building the calculation of periods and modes of free oscillations is required (1). While determining theoretically of these parameters for various buildings one meets certain difficulties (8). In order to derive formulae, well-grounded and simple, practical and theoretical investigations of free oscillations of buildings with load-bearing stone walls were carried out. Tests of such buildings were carried out with electro-dynamic vibrograph and the records were made with the oscillograph (18; 19). In Fig. 7 the records of oscillations of a 6-story building with the variable thickness along the height of load-bearing walls. One of the records presents the second form of oscillations. Theoretically this problem was solved with the consideration of elastic clamped building in the ground. As a result we obtained the formula for the determination of free oscillation periods for such type of a building:

$$T = \frac{9.75\,H}{\alpha_i}\sqrt{\frac{m}{FG\gamma_0}}$$

where H is the height of the building in cm; α_i - is the frequency coefficient which is determined from the diagram in Fig. 8 and which depends on the interrelation of rigidities of the building and the ground; i - is the mode of oscillations; m - is the mass in kg. sec^2/cm; F - is the area brutto of walls in the plan in cm^2; G - is modulus of elasticity of masonry for shear in kg/cm^2 which at low oscillations is equal to 0.4 E and at high oscillations is equal to 0.25 E where E is modulus of elasticity of masonry for bending; γ_0 - is the coefficient taking into account the area of openings in walls. We obtained rather simple formulae which agree satisfactorily with a series of field measurements of periods:

$T = 0.0165\,H$ - brick buildings

$T = 0.014\,H$ - big block buildings

where H - is the height of the building in m.

In Fig. 9 there are given comparisons of measured (Fig. 7) and calculated periods of oscillations of a 6-story building. It is necessary to point out that the difference for the first two modes is equal to 3 percent and 1 percent, respectively.

c) The development of experimental investigations of small models of buildings and structures tested on seismic platforms is to our mind rather an effective means for solving a number of problems concerning designing and building of antiseismic structures.

This paper presnets a brief summary of experimental investigations carried out on small three-dimensional models of interconnections of brick walls of buildings, some types of hydrotechnical structures made of large concrete blocks and earth dam of slope type.

The investigations carried out are some experience of solving certain problems concerning antiseismicity of building and structures by means of experimental investigation of their three-dimensional models.

More than 200 models of various types were tested on a seismic platform with photoelectric program control.

Experiments with models of the interconnection of brick walls (20) were carried out by comparing the behaviour characteristics of two models which were under the action of horizontal oscillations at their base. Both models were equal in their external sizes and materials but differed constructively. The experiments showed that strengthening the construction of walls only in horizontal joints does not provide their stability during horizontal oscillations. The stability and resistivity of wall interconnections increases greatly with vertical reinforcement of the construction (Fig. 10).

The measurements of horizontal oscillation amplitudes on the levels of floors (Fig. 11-12) in the stage of destroing models of interconnections of walls 1 - 4 stories high (21) gave amplitudes on the level of floors which are shown in Fig. 12.

Experiments with two similar models of wall intercommections which are situated on pliable and ridge ground showed that at horizontal oscillations in the base with the ranges of frequency equal to 1.40 - 4.75 c/sec. both models vibrated and the frequency of oscillations was rather close to the frequency of their free oscillations.

The frequency of free oscillations of the model on ridge ground exceeded the frequency of free oscillations of the model on a pliable ground by 42 percent. The pliability of the ground appears to lead to some decrease of the structure free oscillation frequency (22).

Experiments of models of slope-type big massive-fill hydrotechnical structures showed their considerable stability during the action of

horizontal oscillations in their base as compared to the models of similar concrete block masonry structures having longitudinal vertical joints. In these last models considerable damages and destruction caused by the lamination of masonry along vertical joints were observed. Walls of concrete block masonry where concrete blocks span the whole wall width without vertical joints showed the best stability (20).

Experiments with models of construction of embankment walls built of usual concrete block and having vertical joints showed that they do not possess required stability when they subjected to strong horizontal oscillations in the base. The principal cause of their destruction consisted in the lamination of masonry and considerable displacement of concrete blocks leading to their destroying (20).

Models of sheet-piling type of structure with anchor devices and some other types of wall constructions showed the best stability (20).

Experiments with models of an earth dam (20) showed that the principle condition providing the seismic stability of the dam at acceleration range of 0.03 - 0.30 g is the initial porosity which is approximately equal to 36-37 percent for fine and small grain sands which were the building material for the dam model. At such initial porosity slopes 1 : 2 appear to be stable (Fig. 13-16).

As a result of these investigations certain regularities in horizontal oscillations in models of interconnections of brick walls and walls made of regular concrete blocks were elucidated, which ascertain the character of displacements according to the height of the model. Characteristic distinctions of the behaviour of concrete block masonry in aerial and water medium and regularities in changing setting, porosity and slopes in a dam built of sand (of not connected earth materials) were established with their dependance upon time.

The Perspectives of Developing Theories of Antiseismicity and Antiseismic Engineering

As a result of work of a number of Soviet and foreign scientists (J. L. Korchynski, A. G. Nazarov, B. K. Karapetjan, M. G. Ourazbajev, S. V. Medvedev and others (23), M. Biot, G. Housner, K. Kanai and others (9, 10, 26, 27, 28) in the USSR, the USA and Japan the dynamic theory the principle theses of which are stated above was developed to a great extent. Such development was caused by the establishment of some statistic data concerning the movement of soil during an earthquake and consequent reaction of various types of structures, this reaction being characterized by dynamic parametres of structures. It if proved that at an average the value of acceleration which is passed to the pendulum by an earthquake depends upon the period of free oscillation of this pendulum. There is a general statistic regularity according to which maximum accleration of the pendulum noticeably decreases with the increase of its period. The essence of a new dynamic theory of antiseismicity consists in direct usage (while working out methods of structure design) of this new characteristic of

the earthquake process. The new method of design is undoubtedly nearer to reality than all the previous methods and its introducing into Normatives (1) is progressive. However, this method insufficiently and not quite consistantly uses the statistic character of the process (of an earthquake and response of the structure) which is dealt with in the present case.

It stands to reason that at the present time when for characterizing earthquake processes we have only one spectral curve there is no possibility to propose anything better than this method. However, as new statistic data concerning an earthquake process are accumulated a new more perfect theory may be developed (24). Modern science possesses methods which give the possibilities to reveal regularities typical of similar stochastic (non-stationary) processes, therefore the problem is to spread these methods over studying the process of the structure deformations during earthquakes. The purpose of the stochastic theory of antiseismicity is the determination (according to statistic characteristics of an earthquake process) of statistic characteristics of deforming structure elements and the indication of reasonable limitations for them. Further development of antiseismic theory seems to follow this way (25). However, a new dynamic method of design given in the Norms (1) should be perfected and assuming it as basis it is necessary to work out new constructive solutions satisfying seismic conditions to the utmost.

Structures with the minimum possible weight the decrease of which must be carried out at the expense of light roof and floor constructions and diminishing lateral rigidity in the wall plane are the most perspective. The use of plastic and aluminium roof constructions for frame buildings of industrial shops leads to loads approximately equal to 20-25 kg/m^2 instead of 350 - 500 kg/m^2. In such case considerable decrease of seismic force values leads to rather effective decrease of the cost of building, as in a number of cases a wind load may be prevalent in the combination of loads. This means that additional antiseismic measures can be reduced in cost to quite moderate expenditures (29). Still greater economical effect can be achieved for frame buildings taking into account plastic deformations of constructions. As a preliminary analysis shows (13) plastic deformations may be considerable without destroying the frame construction. Undoubtedly working out a method of construction design for seismic action with the consideration of plastic deformations is one of most urgent problem of anti-seismicity. The idea of designing such antiseismic structures in which it is possible during an earthquake to allow large deformations (foreseen by the design and constructing) and even destruction of separate secondary elements if these deformations and destruction do not cause demolition the whole construction must appear to be perspective.

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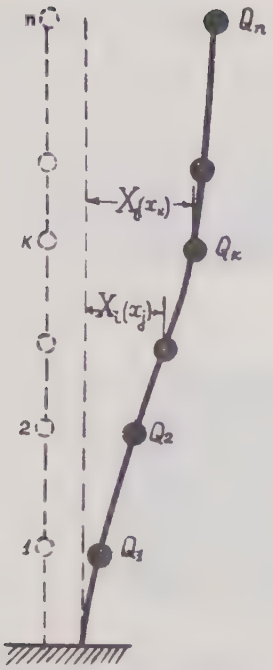


Fig. 1. Design scheme of elastic system with "h" degrees of freedom which is acted upon by a horizontally displacing foundation.

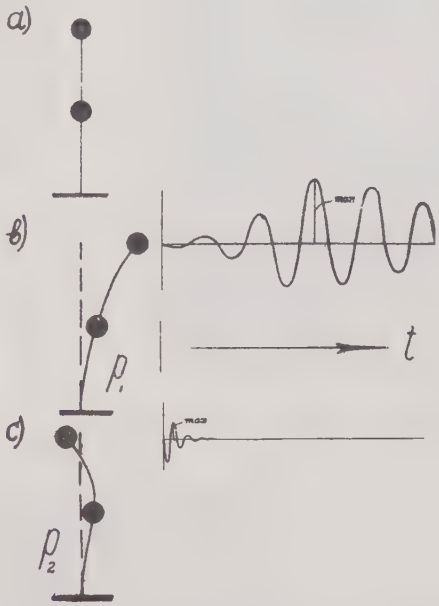
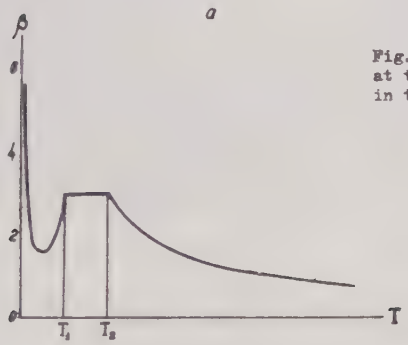


Fig. 2. Deflections of the system with two degrees of freedom at two first modes of free oscillations generated by movement in the foundation of the system.



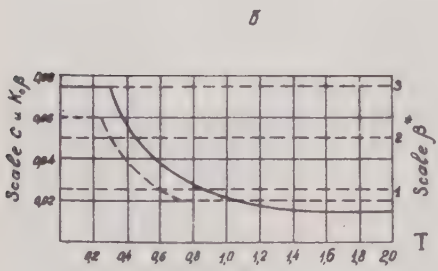


Fig. 3
a) The diagram of of values β for systems with various periods of free oscillations.

b) The comparison of diagrams of β and C coefficients.

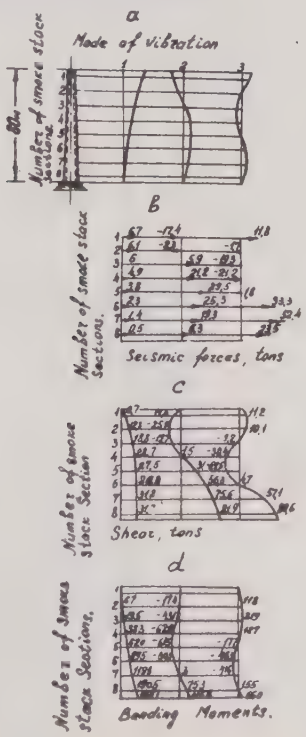


Fig. 4
The design data of seismic foces, shears and bending moments for a reinforced concrete chimney, 80 m high;
a – oscillation modes;
b – seismic forces;
c – shears;
d – bending moments

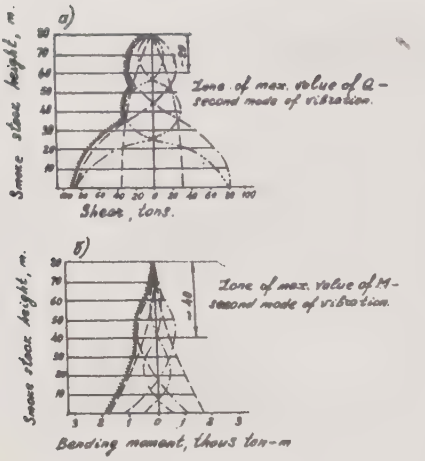

a)
Zone of max. value of Q - second mode of vibration.
Shear, tons.
b)
Zone of max. value of M - second mode of vibration.
Bending moment, thous ton-m
For design according to 1-st second mode of vibration.
For design according to 2-nd mode.
For design according to 3-rd mode.
envelope when designing according to proposed method.

Fig. 5. Envelope epures of forces;
a - shear;
b - bending moment.

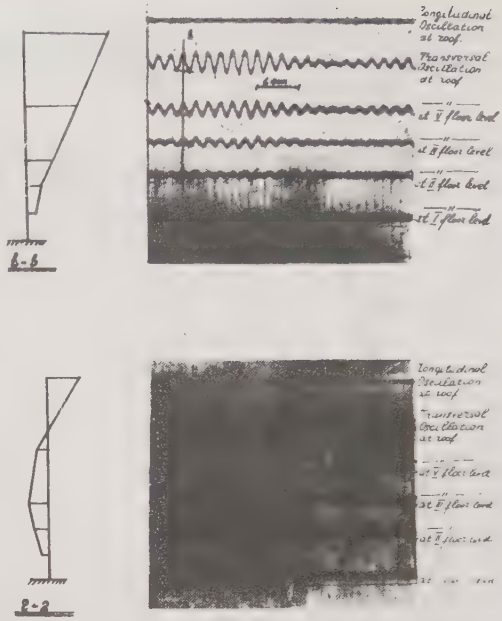


Fig. 7 Records of oscillations of a six-storied buildings and the corresponding modes of oscillations: the first (b-b) and the second (2-2).

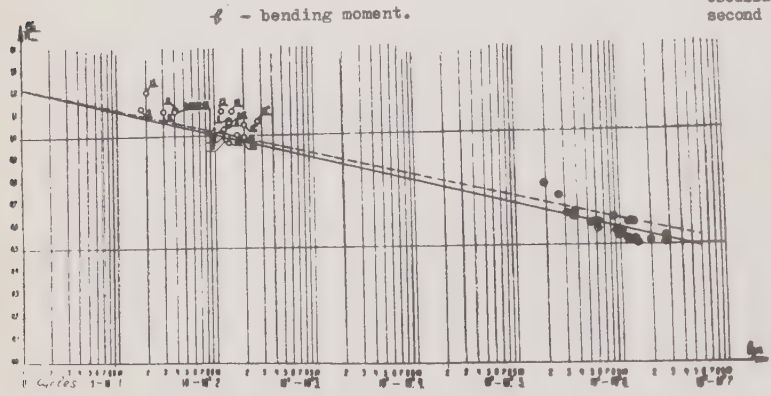


Fig. 6
Lineal relation between the value of destroying /limit/ dynamic stress and the number of loading cycles. The solid line - theoretical relation at asymmetry coefficient equal to zero. The dotted line - theoretical relation at asymmetry coefficient equal to 0.143.
• - experimental data on fatigue
o - experimental data at not numerous repeated loadings

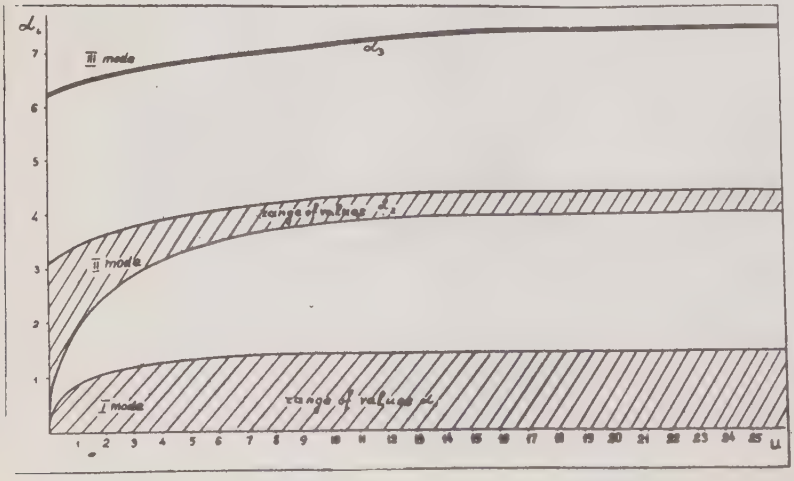


Fig. 8
The diagram: frequency coefficients (αi) for three lower oscillation modes
$U = \frac{K_1}{FG} \frac{K_\varphi}{K_\psi} H \qquad V = \frac{K_1 K_\varphi}{HFG}$
$\frac{K_1}{FG}$ - is the rigidity characteristic of the building;
K_φ - the rigidity of the foundation at turn;
K_ψ - the characteristic of the foundation rigidity at shear shear;
H - the height of the building.

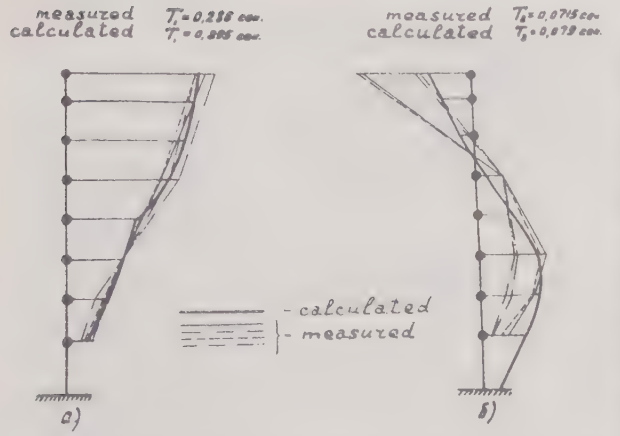


Fig. 9. The comparison of measured and calculated periods and modes of oscillations of a six-storied building with a varying width of walls.

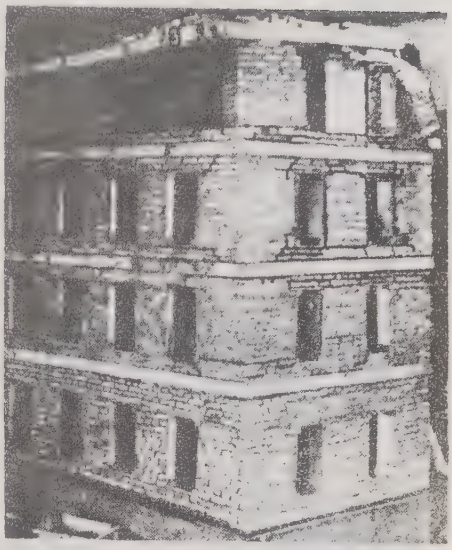


Fig. 11. The character of destruction of a four-storied brick building model.

Fig. 10. Two simultaneously tested models of brick two-storied buildings, one model being reinforced vertically.

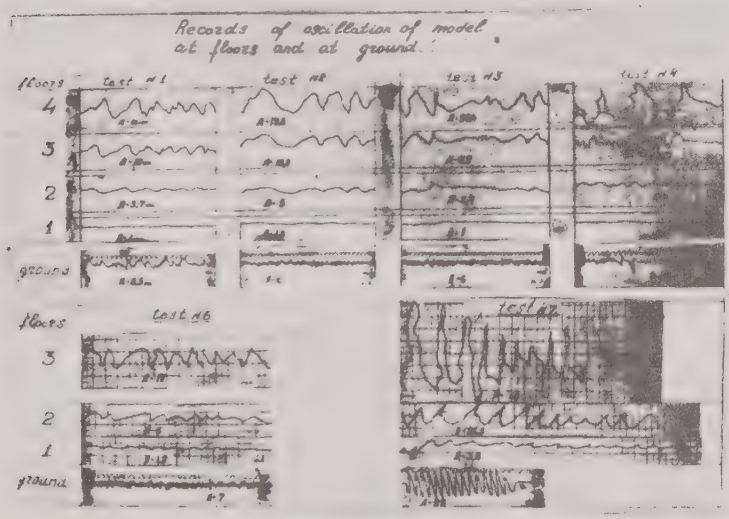


Fig. 12. Vibrograms of recorded oscillations of models on various stories and at the foundation.

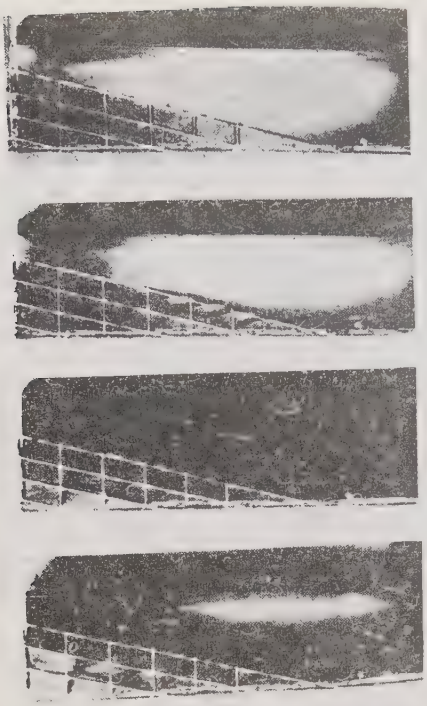
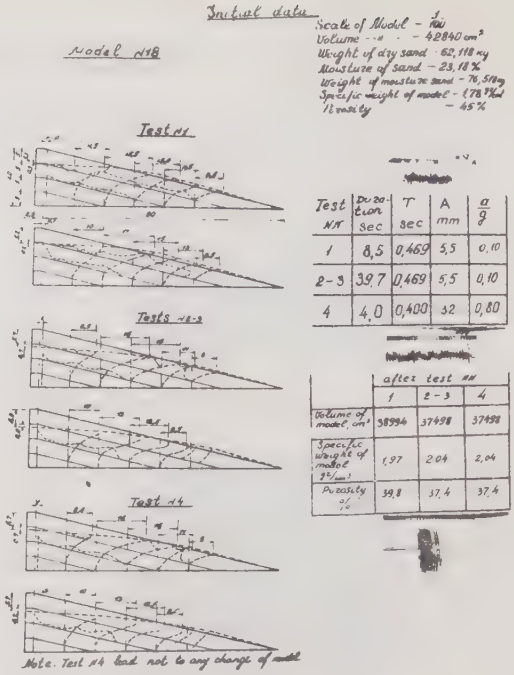


Fig. 13 The charater of deforming the slope 1:4.



Test №№	Duration sec	T sec	A mm	a/g
1	8,5	0,469	5,5	0,10
2-3	39,7	0,469	5,5	0,10
4	4,0	0,400	32	0,80

	after test №		
	1	2-3	4
Volume of model, cm³	38994	37498	37498
Specific weight of model g/cm³	1,97	2,04	2,04
Porosity %	39,8	37,4	37,4

Fig. 14 The same. The dotted line shows the situation of coloured parts of sand after the corresponding oscillations.

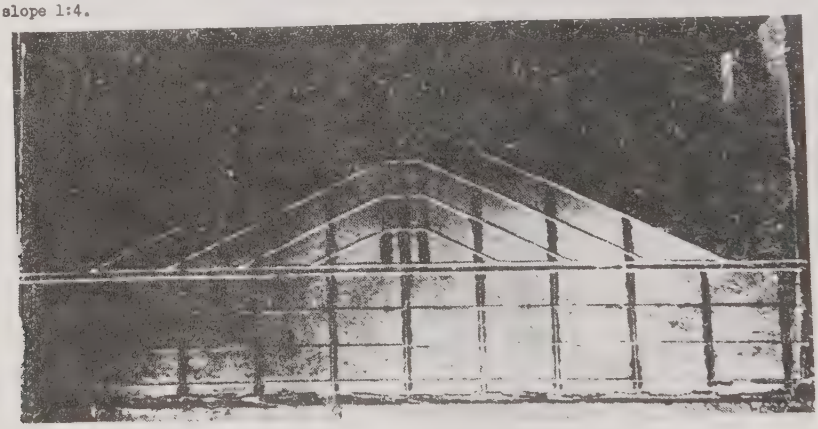


Fig. 15. The model of a dam with slopes 1:2 on soft base at porosity equal to 45 percent.

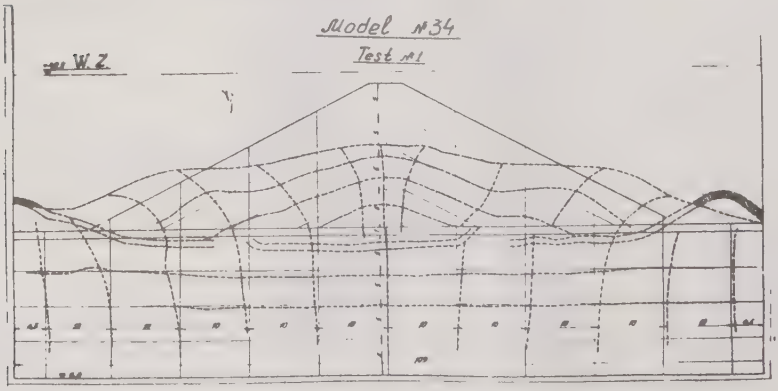


Fig. 16. The character of deforming the model shown in Fig. 15. The dotted line indicates the situation of coloured parts of sand.

Drift Limitations Imposed by Glass

J. G. Bouwkamp[1]
J. F. Meehan[2]

INTRODUCTION

Engineers interested in proper aseismic design have for many years recognized the importance of drift limitations in the analysis of buildings. The reason for this interest is twofold, one to prevent damage, both structural and non-structural, and the other, the disturbing psychological effect on the occupants due to the movements which can cause panic. The California Administrative Code Title 21, which is the building code for the construction of public school buildings in the State of California, has placed in Section 111, the following limitation on drift: "The deflection of vertical-resisting elements due to wind or seismic loads in the plane of the wall shall not exceed one-sixteenth inch per foot of height of the element. Deflection in the plane of the wall from the head to the sill of a glazed opening shall not exceed one-sixteenth inch per foot of height of the opening unless the glass therein is prevented from taking shear or distortion or wire glass is used." This amounts to 0.0052h. Admittedly, this value of one-sixteenth inch per foot of height was not founded on scientific knowledge. It was felt that since Title 21 is directed toward safety and that glass was one of the most hazardous materials in a structure that if any investigation is made it should be directed toward the drift limitation imposed by glass rather than toward the psycological effects.

At the request of several groups, and to obtain this desired information, a research project investigating the behavior of window panels under in-plane forces was performed at the Engineering Materials Laboratory, Division of Structural Engineering and Structural Mechanics, Department of Civil Engineering, University of California at Berkeley.

This paper presents a brief summary of the results of this test program as discussed in detail in the report "Behavior of Window Panels under In-plan Forces", by J. G. Bouwkamp.

The research project was sponsored by the Division of Architecture, Department of Public Works, State of California, and other interested organizations. A complete list of reports resulting from research projects on earthquake engineering subjects as sponsored by the Division of Architecture is given in Appendix I.

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TEST PROGRAM AND PROCEDURE

After giving consideration to the various types of window details used in school construction, the following list of variables were selected:

1. Type of sash (for typical sections see Figure 1)
   - a. Steel
   - b. Aluminum
   - c. Wood
2. Type of Glass
   - a. Double strength: 1/8 inch in thickness
   - b. Crystal: 3/16 inch in thickness
   - c. Plate: 1/4 inch in thickness
3. Bedding mastic
   - a. Soft
   - b. Hard - a gypsum celite plaster was used to simulate hardened aged putty.
4. Clearance of glass in sash
   - a. 1/4-inch clearance all around
   - b. 3/8-inch clearance all around
   - c. 1/2-inch clearance all around
5. Location of wood-screw attachment
   - a. All four sides
   - b. Top and bottom
6. Size of sash
   - a. 2 feet high by 4 feet wide
   - b. 4 feet high by 4 feet wide
     - 1) Single pane
     - 2) Subdivided

c. 8 feet high by 4 feet wide

1) Single pane

2) Subdivided

7. Loading

a. Static

b. Alternate static

c. Impact

The sash were mounted on a 2 by 6 inch wood member and attached by means of wood screws. (See Figure 2.) These wood members were attached to a pin-connected or hinged steel frame, thus allowing the window panels to carry the full horizontal in-plane load applied at the top of this hinged frame. (See Figure 3.) As the window panel deforms under the applied load, the dead load of the hinged frame will introduce additional vertical forces on the panel. A spring was installed to keep the hinged frame in equilibrium in any deflected position.

The following measurements were taken by means of 1/1000-inch mechanical dial gages:

1. In-plan movements of glass panel with respect to sash.

2. In-plan movements of the window panel.

3. Out-of-plane movements of the glass panel (only in the Series No. 1).

4. Movement of the hinged frame.

A total of thirty-three specimens, subdivided into Series I, II, and III were tested under a static load to investigate successively, the effect of the variables. Two specimens two by four feet in size were tested under an alternating-reversible load, inducing one-fourth inch deflection increments in each direction. Four panels were tested under impact loads.

TEST RESULTS

The results of the static-load tests and the comparable theoretical in-plan deflections for the Series I, II, and III are given in the Tables I, II, and III.

In general the action of the steel and aluminum-sash window panels with soft mastic is as follows: the sash deflects as a frame until the sash bears upon the glass at the loaded corner and in the region of the diagonally opposite corner. At this point the glass rotates within the sash frame, thereby permitting considerable additional horizontal movement. When the glass is seated in the two diagonally opposite corners there is very slight additional movement as this additional movement is due merely

to that which would be permitted by the shortening of the diagonal as a result of the buckling of the glass. (See Figure 4)

For a wood-sash window panel with soft-bedding mastic the edge of the glass penetrates into the wood sash and opens the sash frame corner connections, thereby permitting larger displacements than those of the metal-sash.

When the bedding mastic is hard, the movement of the head of the sill is due primarily to the crushing of the hard mastic and of the wood under the sash frame.

Figure (4) indicates the essential configuration of the sash and glass when soft-bedding mastic is used and no rotation of the window panel occurs due to crushing the wood. Figure (5) indicates the configuration when the sash is allowed to rotate within the building frame. Essentially it was learned that for metal-sash window panels the drift limitations imposed by a single pane of glass bedded in soft mastic is merely a geometrical relationship primarily depending upon the glass clearance (c) and the panel dimensions d (height) and w (width), namely:

$$\Delta - \phi\, d = 2\, c \left(1 + \frac{d}{w} \right)$$

In multiple metal-sash window panels a similar expression could be derived in which the total deflection is primarily the summation of the deflections of the individual panes in one vertical line, namely:

$$\Delta - \phi\, d = \lesssim 2\, c \left(1 + \frac{d}{w} \right)$$

Also, an expression determining the drift limitations at failure imposed by a single pane of glass where the bedding mastic is hard, was derived from these test results, namely:

$$\Delta - \phi\, d = F \quad 2\, c \left(1 + \frac{d}{w} \right)$$

where $F = 0.36 - 0.115 \frac{w}{d}$

This expression for F represents good agreement with the test results from Series II, but further tests should be performed to evaluate the factor F more accurately.

It can be shown geometrically that in all of the above quantities the portion $(1 + \frac{d}{w})$ should be further decreased by the quantity $\frac{2c}{w}$; however, this has been neglected due to its insignificant effect on the results.

For soft mastic glazed metal-sash window panels the glass failed locally in the loaded corners. The failure occurred by crushing the glass in one and sometimes in both loaded corners. (See Figure 6&7) A similar type of failure combined with the failure of the sash bars occurred in the multiple-pane metal-sash window panels using soft mastic. (See Figure 8.) For wood-sash window panels the glass failure was more complete because of the higher loads or the contained potential energy, under which these panels failed. This large load was a result of the restraining effect of the

relatively rigid wood stops at both sides of the glass. For hard-mastic glazed-window panels, a buckling failure of the glass took place generally over the total glazed area. The restraining effect of the hardened mastic on the glass panel allowed a considerably higher failure load than for soft-mastic glazed panels, thus causing a generally complete failure of the pane.

The results from the load-reversal tests indicated that the deflections at failure were practically identical to the deflections at failure for identical panels under a static-unidirectional load.

The results from the impact loading tests, in which the load was obtained by swinging a pendulum weight against the head of the hinged frame, indicated that the deflection of the window panel at failure is essentially the same as that under static loads. (See Table IV.) The energy load required to produce glass failure under these impact-load conditions is higher than under static-load conditions. The failure pattern was similar to the soft mastic specimens although not as pronounced.

This investigation has shown that where care is taken to insure clearance for movement of the glass in the sash, a considerable amount of drift can be tolerated before glass breakage will occur. However, where movement is impaired, such as by a hardened mastic or the sash frame, hazard exists for both the building occupants and the public outside the building. When the glass fails, fragments of glass are thrown outward from both sides of the sash. Upon failure of the two by four-foot panels under static-loading glass fragments were found as far as five feet out from both sides. On the eight-foot high by four-foot wide panels the lateral travel of the glass was as far as 25 feet outward from the sash. Due to testing limitations, this type of complete failure of glass during impact loading was not obtained for the hard mastic specimens; however, it is believed that a similar failure pattern and fragment dispersal would be observed as that of the static load tests.

This investigation also indicates that the drift limitation of .0052h is not conservative where the bedding mastic is hard as indicated by specimen numbers A-1-a-$\frac{1}{2}$-h, A-2-a-$\frac{1}{2}$-h, and A-3-a-$\frac{1}{2}$-h. In each case the tested failure movements, $\Delta - \phi$ d, were 0.02 inch less the permitted design movement. The $\Delta - \phi$ d value would apply to the design of a building where the sash is connected directly to building frame and not to an intermediate wood member.

ACKNOWLEDGEMENTS:

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APPENDIX I

AVAILABLE DIVISION OF ARCHITECTURE

RESEARCH REPORTS

The following list of reports of research projects sponsored by the Division of Architecture are available from Printing Division, Documents Section, Sacramento 14, California:

1. "Behavior of a Continuous Concrete Slab Prestressed in Two Directions", by A. C. Scordelis, T. Y. Lin and R. Itaya, price $1.75, plus tax*.

2. "Behavior of Prestressed Concrete Beams at Transfer" by T. Y. Lin, A. C. Scordelis and R. H. May, price $1.75, plus tax*.

3. "Shearing Strength of Reinforced and Prestressed Concrete Lift Slabs", by T. Y. Lin, A. C. Scordelis and R. Itaya, price $1.50, plus tax*.

4. "Radiographic Inspection of Reinforced Grouted Brick Masonry", by J. R. Benjamin and H. A. Williams, price $1.25, plus tax*.

5. "Lateral Load Tests on Reinforced Grouted Masonry Shear Walls" by R. R. Schneider, price $1.75, plus tax*.

6. "Frictional Effects in Composite Structures Subjected to Earthquake Vibrations", by L. S. Jacobsen, price $1.75, plus tax*.

7. "Action of Timber Structures Subjected to Lateral Loads", by C. E. Troxell and V. Bertero, price $1.75, plus tax*.

8. "Behavior of Window Panels Under In-Plane Forces", by J. G. Bouwkamp, price $1.75, plus tax*.

The following reports are available without charge from the Forest Products Research Center, State of Oregon, Box 571, Corvallis, Oregon:

1. "Lateral Tests on a Full-Scale Roof Diaphragm Sheathed with 2 by 3 inch Lumber", by R. A. Currier, Report No. T-10, October 1954.

2. "Lateral Tests on 12 by 60 foot and 20 by 80 foot Lumber Sheathed Roof Diaphragms", by J. A. Johnson, Report No. T-12, October 1955.

3. "Lateral Test on a 20 by 60 foot Roof Diaphragm with Stapled Plywood Sheathing", by J. W. Johnson, Report T-13, December 1955.

4. "Strength Tests on Full-Scale Plywood Sheathed Wall Section", by R. A. Currier, Report T-15, February 1956.

* If purchaser is California resident, add 4% sales tax.

5. "Racking Tests on Wood Wall Sections with Mullions", by A. D. Hofstrand, Report No. T-17, September 1956.

6. "Lateral Tests on Full Scale Cable Roofs with Lumber Sheathing", by J. W. Johnson and C. H. Burrows, Report No. T-19, October 1956.

The following report is a digest of several papers and is available without charge from the United States Forest Products Laboratory, Madison 5, Wisconsin:

"Diaphragm Action of Diagonally Sheathed Wood Panels", by D. Y. Doyle, Report No. 2082, November 1957.

Nomenclature

A - aluminum sash

S - steel sash

W - wooden sash

1 - panel size 4' x 2'

2 - panel size 4' x 4'

3 - panel size 4' x 8'

p - panel subdivided into 2' x 4' panels with the 4' dimension horizontal

c - panel subdivided into 2' x 4' panels with the 4' dimension vertical

a - panel attachment all around

b - panel attachment to head and sill only

1/2 - 1/4" clearance all around between glass and sash or 1/2" total clearance

3/4 - 3/8" clearance all around between glass and sash or 3/4" total clearance

1 - 1/2" clearance all around between glass and sash or 1" total clearance

s - standard soft bedding mastic (Fuller Steel DAP 1012)

h - hard bedding mastic substitute (gypsum-celite)

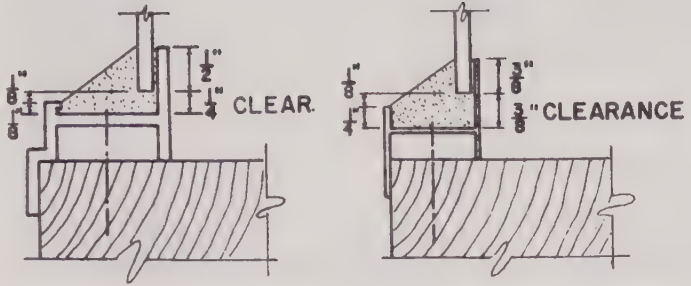
d - vertical sash dimension

w - horizontal sash dimension

e - clearance between glass and sash

ϕ - rotation of sash frame

Δ - total displacement of head of sash with respect to sill



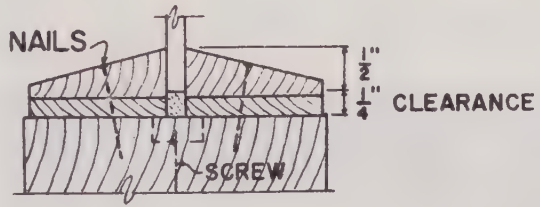


Fig. 1 Sash Details

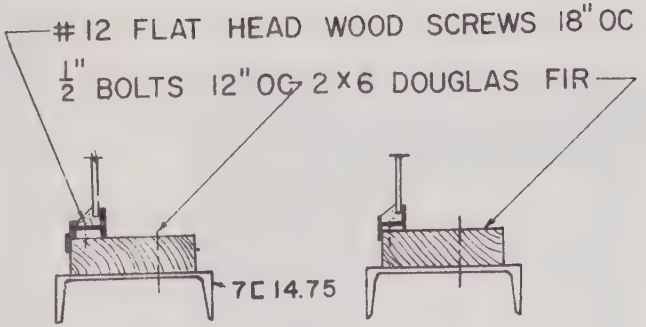


Fig. 2 Mounting of Sash to Testing Frame

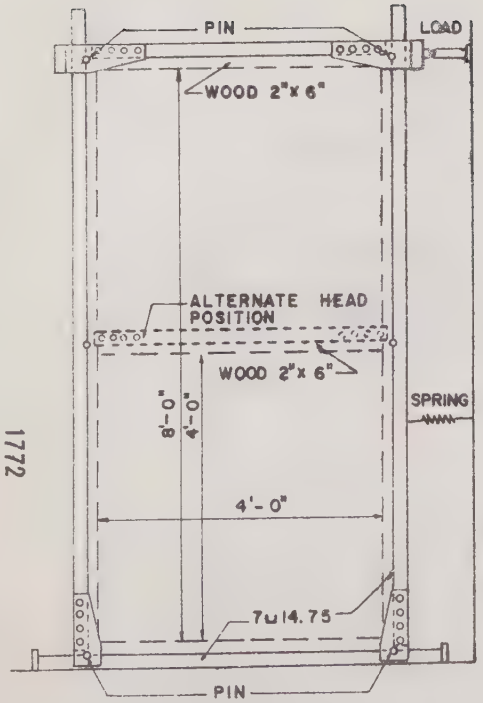


Fig. 3 Hinged Loading Frame for 4'×4' and 4'×8' Window Panels

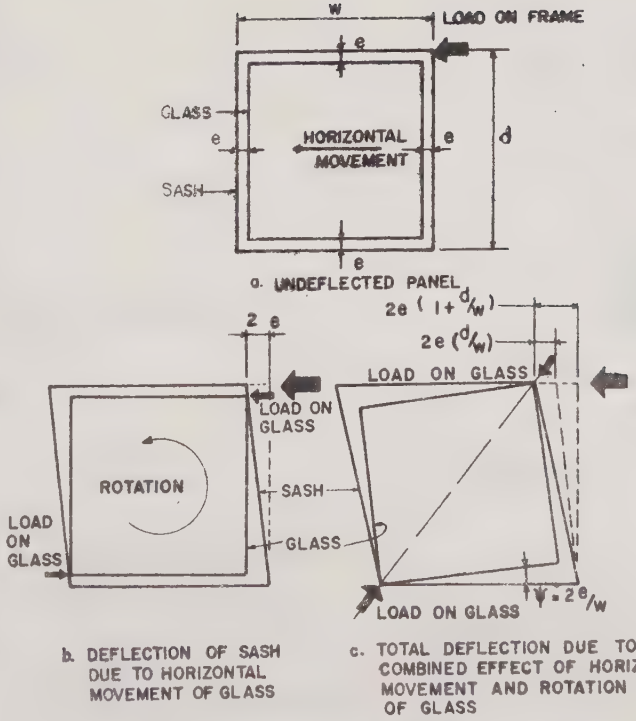


Fig. 4 Displacement of Glass within Sash Soft Mastic Bedding (Zero Rotation ø of Sash)

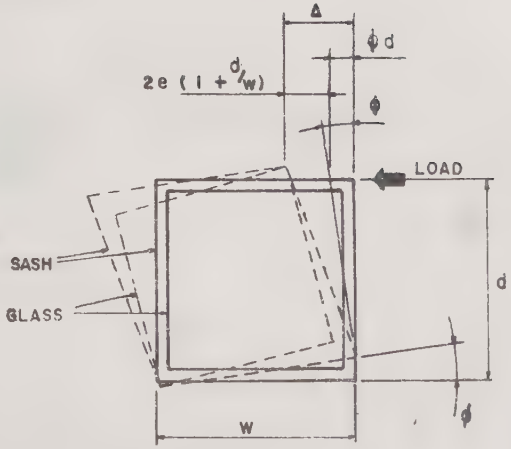


Fig. 5 Displacement of Glass within Sash Soft Mastic Bedding (Rotation ø included)

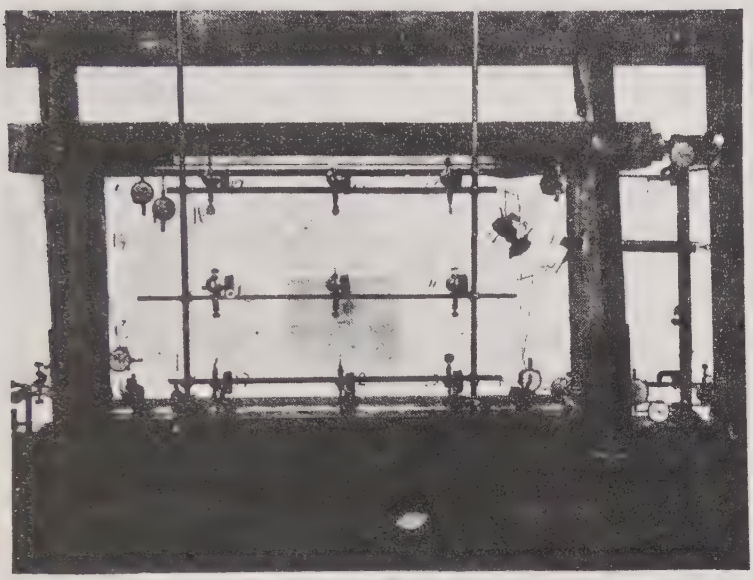


Fig. 6 General Dial Arrangement 4' x 2' Aluminum Sash
Glass Failure Upper Right & Lower Left Corner

Fig. 7 4' x 8' Wood Sash

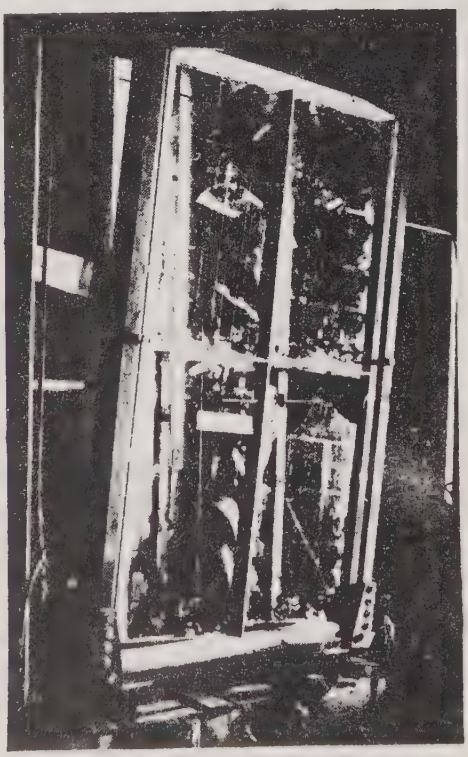


Fig. 8 4' x 8' Vertically Subdivided Aluminum Sash

TABLE I: RESULTS OF SERIES I

OBJECTIVE: Study of the Effect of Sash Material and Panel Attachment

Specimen	Deflections at Failure Δ in.	ϕd in.	$\Delta - \phi d$ in.	Failure Load P lb.	Theory $\Delta - \phi d$
A-1-a-1/2-s	0.85	0.03	0.82	2100	0.82
A-1-b-1/2-s	0.82	0.07	0.75	1940	0.75
A-3-a-1/2-s	1.46	-0.25	1.71	2150	1.50
A-3-b-1/2-s	2.36	0.51	1.85	1620	1.50
S-1-a-3/4-s	1.19	0.00	1.19	2680	1.13
S-1-b-3/4-s	1.19	0.06	1.13	2770	1.13
S-3-a-3/4-s	2.26	0.10	2.16	1720	2.25
S-3-b-3/4-s	1.84	0.28	1.56	1380*	2.25
W-1-a-1/2-s	1.41	0.06	1.35	3440	
W-1-b-1/2-s	2.00	0.05	1.95	3900	
W-3-a-1/2-s	3.52	0.34	3.18	3340	
W-3-b-1/2-s	3.83	0.47	3.36	3430	

* Failure due to vertical compression of glass panel and early buckling before total in-plane movement was reached.

Key

A-1-a-1/2-s = Aluminum sash, 4 ft. horizontal x 2 ft. vertical, attachment all around, 1/2 inch total clearance, soft bedding mastic.

TABLE II: RESULTS OF SERIES II

OBJECTIVE: Study of the Effect of Panel Size, Glass Clearance and Hardness of Mastic

Specimen	Deflections at Failure Δ in.	ϕd in.	$\Delta - \phi$d in.	Failure Load P lb.	Theory $\Delta - \phi$d
A-1-a-1/2-s	0.81	-0.02	0.83	945	0.75
A-1-a-1-s	1.36	0.00	1.36	1720	1.50
A-2-a-1/2-s	1.09	0.28	0.81	4660	1.00
A-2-a-1-s	1.88	0.09	1.79	2400	2.00
A-3-a-1/2-s	2.06	0.37	1.69	2030	1.50
A-3-a-1-s	2.80	0.00	2.80	860	3.00
A-1-a-1/2-h	0.16	0.06	0.10	3850	0.10
A-1-a-1-h	0.27	0.10	0.17	5700	0.18
A-2-a-1/2-h	0.48	0.25	0.23	6420	0.24
A-2-a-1-h	0.67	0.24	0.43	6660	0.49
A-3-a-1/2-h	1.01	0.53	0.48	3550	0.46
A-3-a-1-h	2.20	1.38	0.82	6750	0.93

TABLE III: RESULTS FROM SERIES III

OBJECTIVE: Study of the Effect of Subdivided Panels

Specimen	Deflections at Failure (Δ) in.	(ϕb) in.	($\Delta - \phi$b) in.	Failure Load P lb.	Theory ($\Delta - \phi$b) in.
A-2-p-a-1/2-s	1.82	0.04	1.78	770	1.50
A-2-p-a-1-s	3.02	0.02	3.00	1190	3.00
A-3-p-a-1/2-s	4.31	-0.16	4.47*	1200	3.00
A-3-p-a-1-s	6.06	-0.40	6.46	1300	6.00
A-2-c-a-1/2-s	1.60	0.07	1.53	1570	1.50
A-2-c-a-1-s	2.76	0.06	2.70	1030	3.00
A-3-c-a-1/2-s	2.49	0.00	2.49	860	3.00
A-3-c-a-1-s	5.41	-0.30	5.71	1200	6.00

* Actual failure of first cracking of the glass is not recorded. At failure of 1200 lb. many corners were cracked. This may be the reason for excessive deflection as given in the table.

TABLE IV: IMPACT LOADING

Panel	(Δ) in.	Failure Energy in. lbs.
A-2-a-1/2-s	1.00	1800
A-2-a-1/2-s	1.36	3000
A-2-a-1-s	1.95	6000
A-2-a-1-h	0.75	7800 *

* Actual failure of glass not obtained.

DISCUSSION

H. M. Engle, Consulting Structural Engineer, U. S. A.:

As the speaker has brought out in his paper, control of glass breakage is important in earthquakes, I had one experience in San Francisco, California many years ago. Several panes of glass were broken out of a building 20 stories up. The glass showered down on the street, and it whistled like a bullet as it struck the pavement. Fortunately no one was hit. A large piece landed about 5' from me. If I had been hit it would have been fatal I am sure. From the standpoint of the hazard control of glass breakage in tall buildings in Earthquake is of importance.

J. F. Meehan:

I thoroughly agree with you that precautions must be taken to prevent glass breakage due to drift from seismic forces or wind in the design of not only tall buildings, but of all buildings. This project has indicated that by giving drift proper considerations, glass damage can be essentially eliminated. By simply adding a cushion of wood around the metal sash in a rigid buiding where the bedding mastic is hard increases the deflection at failure of the glass from 160% to 210% (See Table II Specimen Ala$\frac{1}{2}$h to A3alh). I feel reasonably certain that if a flexible gasket of some type (around the glass) could be developed, using a metal stop to seal the glass from the weather, and without a wood cushion, the large movements indicated in Table II Specimens Ala$\frac{1}{2}$s to A3als could be used as a design basis to prevent glass breakage. Where the sash is anchored directly to the concrete of a rigid building, the 1/16" per foot of height of opening should be reduced.

E. Arze, University of Concepcion, Chile:

1. During the recent earthquakes in Chile it was observed that, in some cases, all the glasses of certain windows will fall to the same side. In your tests, did the glass fall to one or both sides?

2. Did you run any test applying forces as vibrations perpendicular to the plane of the window?

J. F. Meehan:

1. The glass dispersal was to both sides of the sash in these tests. Generally, more glass fragments were found on the side with the soft mastic because this mastic was less rigid than the metal stop of the sash. It is roughly estimated that 60% of the glass went to the mastic side of the sash and 40% to the metal stop side. Where the stops were of equal rigidity, such as the wood sash, the glass dispersal was about equal to each side.

2. We made no tests where the load was applied perpendicular to the plane of the glass. Glass and mullion manufacturers have done a considerable amount of work with static loading in this direction. Glass companies and building codes provide recommendations of permissable glass areas for each thickness and type of glass. There are several recommendations, although not found in building codes, for mullion stiffness in the direction perpendicular to the glass. These are:

a) $\frac{L}{240}$ for wind pressure of 20 pounds per square foot.

b) $\frac{L}{175}$ for wind pressure of 10 pounds per square foot.

c) Glass and frame to resist 35 pounds per square foot, with no permanent set in the framing.

GUARANTEEING ANTISEISMICITY OF BUILDINGS ON THE BASIS OF INVESTIGATIONS CARRIED OUT IN THE USSR

By

K.S. Zavriev*

Systematic investigations on problems of antiseismicity of buildings in the USSR date from the second quarter of the 20th century, i.e. they were begun soon after Gori and Leninakan earthquakes. At the same time a progressive dynamic theory of antiseismicity was suggested, this theory was developed later on and was assumed as a basis for designing aseismic structures. Simultaneously aseismic constructions for buildings and other structures were developed. The present paper gives an account of the present state of investigations which are carried out in the Soviet Union on aseismic constructions in buildings.

In the first phase of a destructive earthquake sharp impact causes damage of rigid parts of the building. In the second phase long seismic waves shatter the building and complete the destruction which begins in the first phase. This is an approximate way of the behaviour of a building during an earthquake.

The structure may be considered as a bar whose lower end is clamped in the ground and upper end is free only in those cases when either its transverse horizontal dimensions are small enough in comparison with the height of the structure or when additional constructions guarantee unalterability of horizontal sections. The first case includes structures of tower-type, i.e. towers, chimneys, lighthouses, siloes, etc.; the second case includes high non-skeleton buildings with thickly set walls and partitions closely connected between themselves and with floors.

However, the most part of the buildings erected in seismic regions should be regarded as boxes which deform not only in vertical planes but in horizontal planes as well. The walls perpendicular to the direction of the impulse behave as slabs connected along the contour with the walls and floors parallel to the direction of seismic forces. The walls parallel to the direction of the impulse are chiefly subjected to bending or shear depending on the correlation of their face dimensions-- vertical and horizontal. These walls get horizontal loads from the walls perpendicular to them and from floors.

For a long time the only type of capital buildings in the seismic regions of the USSR was the type with stone walls (chiefly with brick solid walls) and wooden floors. Later on wood in floors was gradually substituted by reinforced concrete and subsequently, with the development of built-up concrete and reinforced concrete construction, the erection of walls of large concrete blocks and even of reinforced concrete panels whose dimensions were designed for a room was put into practice. Lately there appeared a new method of assembling buildings of ready-made three-dimensional cases each case presenting a room. At the present time buildings with solid stone or brick walls still predominate in seismic regions but side by side with

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them buildings of large blocks and large panels and buildings of ready-made rooms are being erected on ever growing scale. Therefore the present paper considers the problems of guaranteeing antiseismicity for all these types of buildings.

1. Building with Solid Stone Walls

It is well-known that with the increase of flexibility of the whole construction the influence of seismic forces decreases. At the same time in order to avoid the breach of connection between parts of constructions it is necessary to get rigid interconnections between them. Therefore the main principle of reinforcement against an earthquake is reduced to bringing separate parts into an unalterable connection by means of ties, at the same time providing as large as possible rigidity of the structure on the whole. Rigid interconnections by themselves experience increased forces, which are conditionally supposed to be fivefold comparing to exiting effects, but it eliminates the possibility of the breach of the building integrity. Guaranteeing the strength of ties causes quite moderate rise in cost of the building as their total volume is small. The most vulnerable parts of the building are projecting elements whose masses are rather small in comparison with the mass of the whole building, i.e. projections of gable walls, chimneys, pediments and parapets. Therefore building such parts should be avoided and in cases when it is inevitable they should be reinforced.

The walls perpendicular to the direction of seismic impulse behave as slabs supported along the contours upon the walls perpendicular to them and upon the floors. If the former walls are weakened by window or door apertures they should be designed as vertical beams (piers) or horizontal beams (seals). Calculations show that if the dimensions are usual sufficient resistivity of walls to such bending is gauranteed in case they are well connected with walls of another direction at places of abutting or intersection. But non-bearing walls are especially endangered as during an earthquake they may be torn away from longitudinal walls. As a proved measure of interconnecting walls we use reinforced concrete belts (girders) extending continuously along all walls at definite levels. The higher is the seismicity of the region the thicker such belts are set. The masonry of walls should be well connected with belts. If floors are securely connected by means of special members with the walls which support beams and with the walls parallel to the latter, the necessary in reinforced concrete belts falls away. It is especially simply achieved at reinforced concrete floors, concreting being made on site (monolithick floors).

The walls parallel to the direction of seismic forces behave as beams clamped into the ground with their lower end and subjected to the action of horizontal seismic forces corresponding to the masses of the walls designed and to the masses of those parts of floors and walls of another direction which load the wall being considered. Under the action of these forces the wall works to bending and shear and depending on the relation of the wall height to its length either bending or shear prevails. In seismic regions we chiefly deal with buildings with a limited number of storeys, i.e. with buildings in which the walls parallel to seismic forces more often work to shear; this is testified by sloping cracks directed at an angle close to 45° (Fig. 1).

If the wall parallel to the direction of seismic impulse is weakened by apertures, horizontal forces are distributed among piers proportionally to their rigidities. Therefore broad piers are especially much loaded. On the whole they are damaged at the expense of large values of seismic forces acting upon them and at the expense of higher dynamical coefficients which as it is well known rise with the increase of the rigidity of a structure. It should be noted that broad piers are chiefly damaged by shearing forces (sloping cracks) and narrow piers are damaged more often by bending moments with horizontal cracks along the pier edges (Fig. 2).

It is most expendient to design walls with piers of equal width. In such cases seismic forces are distributed among them equally. The character of pier damage essentially depends on the location of the earthquake epicentre with respect to the building. If the epicentre is located nearby, considerable values of vertical component seismic forces promote the development of sloping cracks on broad piers. If the epicentre is far, the influence of bending prevails and on narrow piers horizontal cracks develop for the most part.

The corners of buildings appear to be in a hard state (Fig. 3).

The rigidity of edge piers, the tendency of adjoining walls to tear off from each other and asynchronomity of oscillations in meeting walls are of importance here. It confirms the necessity of the arrangement of reinforced concrete belts or other ways of wall interconnecting.

It is clear from all stated above that effective measures for guaranteeing the safety of buildings with solid stone walls are as follows:

1) the arrangement of reinforced concrete belts or connecting floors with the walls of both directions;

2) the even distribution of apertures along the length of the wall.

3) the establishment of limit distances between walls and limit height of the story.

2. Buildings with Skeleton Walls

Experience shows that damage of reinforced concrete frames is often characterized by forming plastic hinges in the joints of the frame. Therefore joints should be strengthened by rubbles and reinforced accordingly. Besides, insufficient cross-section or inadequate reinforcement cause the damage of the lower ends of the columns, the concrete reinforcement becoming bared and buckled. Then, in order to avoid falling out the filling it is necessary to tie it firmly with elements of the frame. The frame should be sufficiently rigid, otherwise brittle masonry of the filling which is unable to follow deformations of flexible frame elements will be subjected to considerable forces in its plane and gradually will fail, especially if it is weakened by apertures.

If the filling is separated from the frame, if it is located on the outside and is supported by the foundation beams, the safety of such "self-bearing" walls depends on the character of connection between them and the

columns of the frame. In such cases a through joint between the walls and the frame is necessary in order to avoid mutual impacts; the connections between them should be flexible.

3. Buildings of Large Blocks

Earthquake-proofness of walls erected of large blocks is provided by constructive solutions analogous to constructive solutions for stone (brick) walls. It is vitally important to secure reliable bond between blocks and mortar. It is achieved by placing blocks under pouring which eliminates the possibility of damaging the mortar at placing blocks to the previously laid mortar. Blocks are mounted dry on wooden wedges making clearance under blocks which corresponds to the thickness of the joint. After the adjustment of a row is completed horizontal and vertical joints are shut from the outside and pouring with mortar under same pressure is effected. As to the floors, for achieving monolithic system they are connected with each other on each side by keys (Fig. 4).

Reinforcement of two adjoining blocks in keys is connected with welded-on metal plates or clips. Rods of cross-section reinforcement are placed close to the keys. Such interconnections bring all floor slabs into monolithic bond and, on the one hand, they guarantee earthquake-proofness of the system and, on the other hand, they guarantee vertical rigidity of floor since all of it works as a single three-dimensional system. It should be noted that, as experience showed, in case of placing floor slabs without making them monolithic longitudinal cracks appear in the ciling plaster along the lines of slab boundaries.

4. Buildings of Large Panels

Buildings of large panels are built with or without a frame. Among frame buildings of large panels the buildings with full frame of columns and girders rigidly connected with each other in the joints are most earthquake-proof. At the same time in order to eliminate the possibility of brittle destruction at rigid connections it is necessary to apply dispersed reinforcement which includes a large number of thin rods. This makes possible the development of plastic flow and the increase of the structure resistivity to short-timed action.

Among the buildings without frame the buildings with transverse bearing walls are preferable for seismic regions. They have some advantages over frame buildings.

For increasing their earthquake-proofness the following supplementary measures should be adopted:

1) the shape of buildings should be rectangular, in the plan, in case of complex shape they should be devided by antiseismic joints into rectangular sections;

2) the walls should be continuous at full length and width of the building;

3) the walls should have smooth surface without projections and be

surmounted only by a small eave and a low parapet;

4) the length and width of the panel should be as large as possible at the expense of its thickness as that will reduce the number of assembly joints presenting the most vulnerable places;

5) a flat roof should be preferred as it results in a rigid membrane securely connecting the walls.

For the last time there spread buildings by the system of engineer Lagoutenko; their bearing transverse walls are formed of thin panels with bulges along the contour; the panels support each other at the ends and consequently work to bending but not to compression. When analysing this scheme from the point of view of its earthquake-proofness we can notice its following advantages:

1) small weight due to the use of thin-walled vertical and horizontal panels;

2) monolithic connection of slabs, serving as the filling of the frame, with bulges along the contour forming the frame after assembly.

3) small loading of the partitions (only their own weight and the weight of one story floor).

The shortcoming of the scheme is the necessity of careful making monolithick joints and small rigidity in longitudinal direction.

The most perfect scheme with respect to earthquake-proofness is the schome with bearing longitudinal and transverse walls and partitions working to compression. In this scheme considerable reduction of free span of the walls is achieved. If the materials are effective and the methods of making vertical panels are progressive (panel-casing method and vibrorolling method) close arrangement of bearing vertical elements of buildings does not cause the increase of the general weight. Such building possesses considerable three dimensional rigidity, as it consists of closed invariable cases, each being made of 4 partition panels and a floor panel. The behaviour of floors improves as within the limits of the room they present slabs supported along the whole contour which increases their rigidity and lightness. High earthquake-proofness of such scheme by itself eliminates the necessity in special aseismic measures.

5. Buildings of Three-Dimensional Elements

The most progressive scheme of built-up reinforced concrete buildings is the scheme of three-dimensional elements presenting ready-made room blocks. This scheme is a good solution for seismic regions as well. Buildings consisting of three-dimensional elements whose width is equal to the width of the building are especially earthquake-proof. Such schemes are particularly tempting due to the fact that for installation of built-up elements it is possible to use gantry-cranes moving along the tracks laid on the outside along the whole length of the buildings. Such portal-cranes are considerably lighter of the usual tower cranes with the same lifting power, especially if the places of suspension of built-up element are fixed close to supports

of the crane beam (Fig. 5). High earthquake-proofness of such buildings is caused by the fact that they consist of light and rigid three-dimensional elements which behave as box girders.

However, separate built-up elements should be mutually and securely connected.

6. Interconnections of Elements in Buildings of Large Panels Erected in Seismic Regions

The Institute of Building of the Georgian Academy of Science worked out constructions for interconnecting elements of large panel buildings. When the seismicity is equal to 7 free joint of the wall is fulfilled as it is shown in Fig. 6.

Loops located in the limits of the slot are let out of the panel. The sense of such interconnection consists in the fact that the concrete in slats does not need by itself the strengthening by means of reinforcing. But the bond of fresh concrete and old concrete is a vulnerable place and the reinforcement should strengthen precisely this bond. For filling the slat with concrete there is no need in timbering.

At seismicity equal to 8 and 9 such connection between wall panels is already insufficient. It is necessary to have such design of connection which would strengthen the resistance of the canal filling to rupture. The design presented in Fig. 7 meets the requirements. Here reinforcement outlets outstep the limits of slats to get them overlap each other during assembling. Four vertical rods are inserted into the loop which forms, at this time after this slats are poured with concrete.

While making such panels it is necessary either to leave slits in moulds to let the reinforcement out or to insert loops in bent state and straighten them later, in the ready panel.

The corners of the building require especially large strengthening. Interconnections in corners for seismicity 7, 8 and 9 are shown in Fig. 8 and 9. The same loopes which have just been mentioned are used with the strengthening consisting of vertical rods but on top there are additional fastenings in the form of welded laid parts. Fig. 10 shows the interconnection at the place of the adjoining of internal and external walls, Fig. 11 shows the interconnections at places of the crossing of internal walls. In both cases loopes are arranged at different levels and are somewhat shifted in relation to other loops meeting with them in the plan.

The connection of floors with the external wall is provided by constructions depending on the thickness of the wall. Floor panels not less than 35 cm thick are supported to the depth of 25 cm without fastening to the wall. When the wall width is less the connection is achieved by welding of laid parts, and the projections of wall panels should be strengthened by vertical reinforcement (Fig. 12).

The supporting of floor panels onto the internal walls is carried out directly at the wall thickness of 26 cm which lets the depth of supporting be equal to 12 cm and thickness of joint 2 cm. The connection between wall

panels (of the given and the next storey) is carried out with the help of vertical rods (Fig. 13).

In order to achieve this into vertical canals formed between the panels are inserted metal frames (60-80 cm long) consisting of 4 rods. It should be noted that the half-length of the frame is placed into each panel. Previously to the mounting of the frame the canal of the given assembled story is poured with concrete up to the level 30-40 cm lower of the upper edge of wall panels. After this a ready frame is inserted into the canal. The diameter of the frame rods is adopted 10 mm for seismicity 7-8 and 12-14 mm for seismicity 9.

All above mentioned types of interconnections bringing elements of large-panel buildings into invariable mutual connection thus guaranteeing their earthquake-proofness are to a certain extent necessary for non-seismic regions. Therefore the strengthening of large panel buildings of a normal height of an order 5 storeys against the action of seismic forces does not result in considerable increase in costs and complication in their erecting. For buildings more than 5 storeys high it is necessary to work out methods of more essential strengthening. The working out of these methods reflected in Fig. 10 and 11 brings us to the recommendation of buildings with "frame without timbering" in which basic interconnections are carried out according to Fig. 14. The pouring of canals with concrete does not require any timbering and after pouring is completed the frame is formed which is securely connected with panels serving as the filling.

Fig. 1

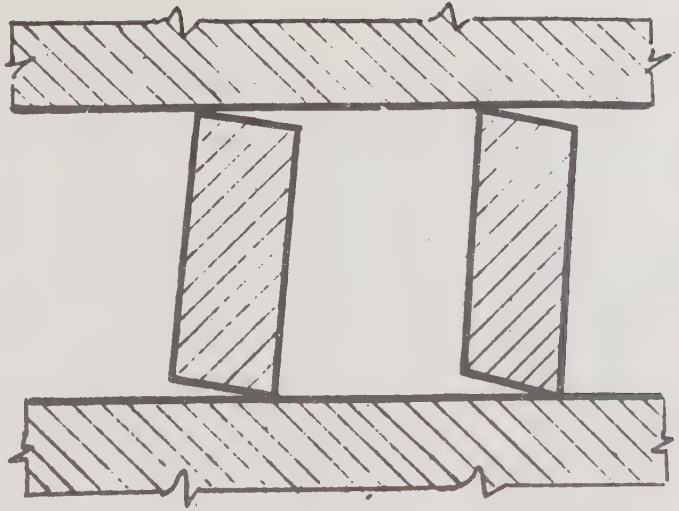


Fig. 2

Fig. 3

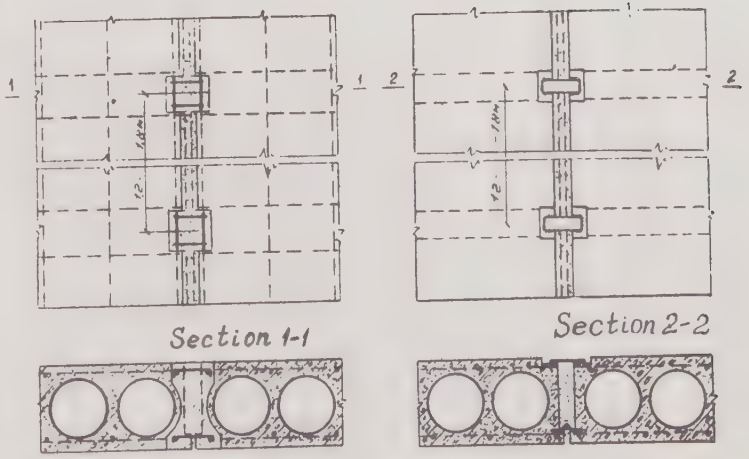


Fig. 4

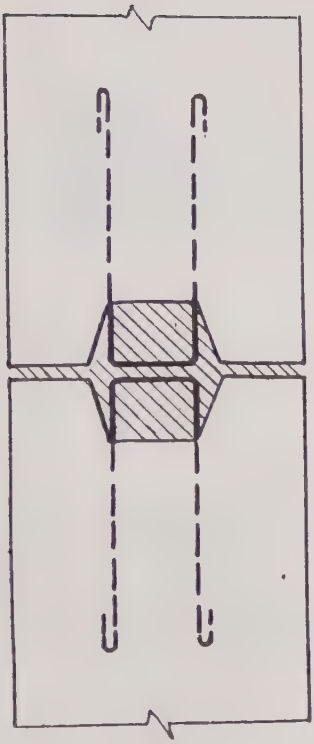


Fig. 6

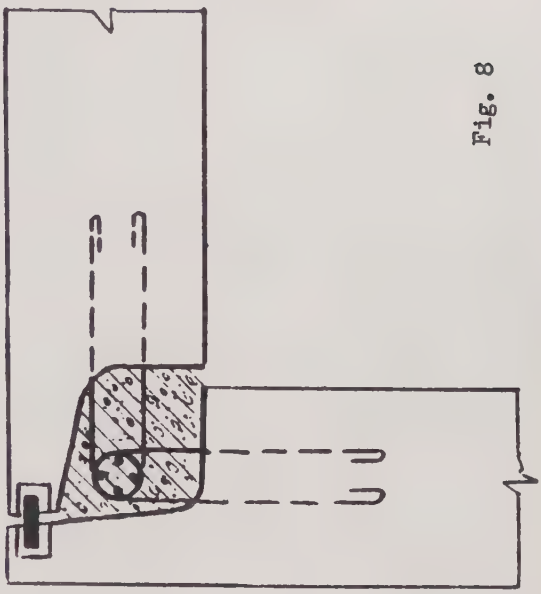


Fig. 8

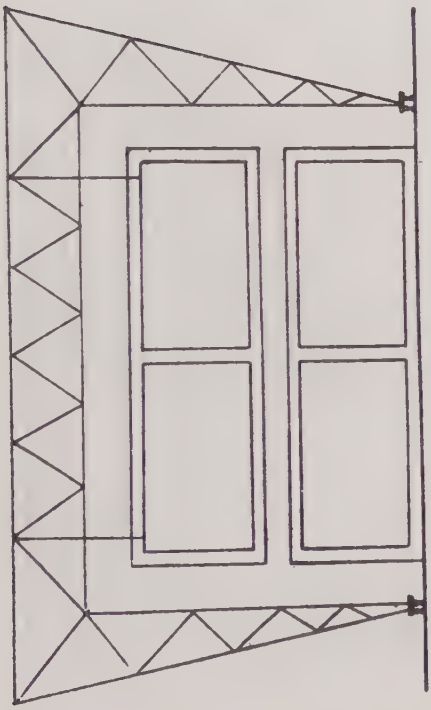


Fig. 5

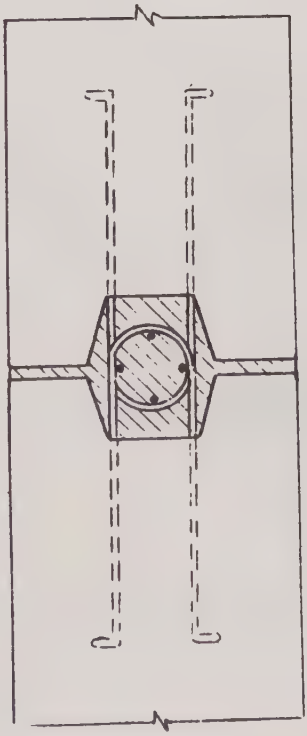


Fig. 7

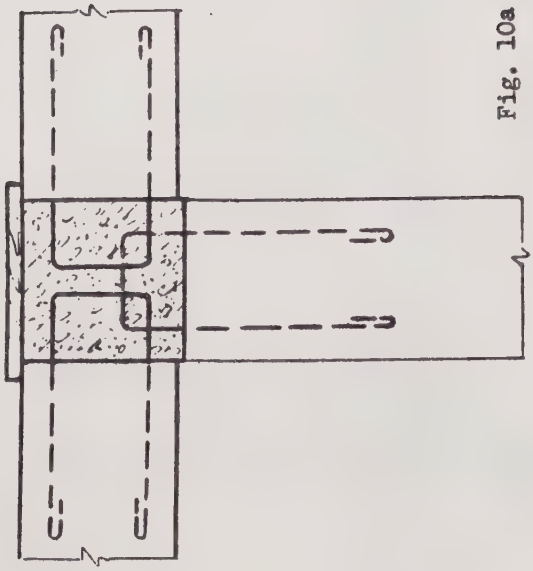
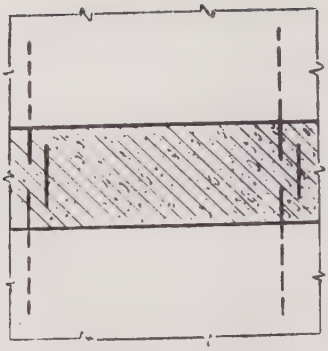


Fig. 10a



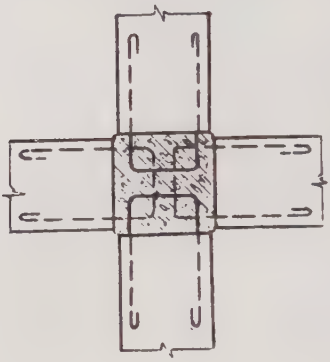


Fig. 11

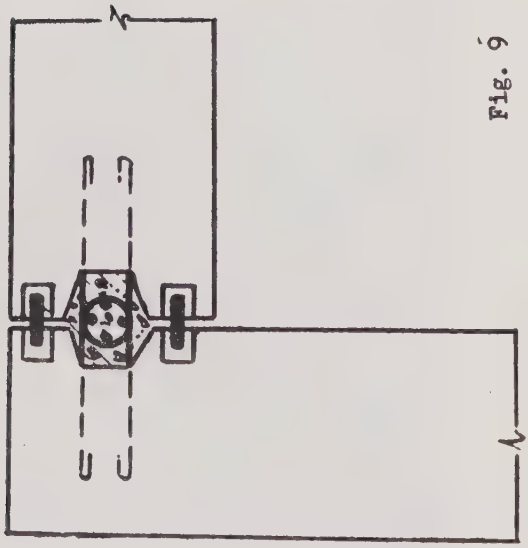


Fig. 9

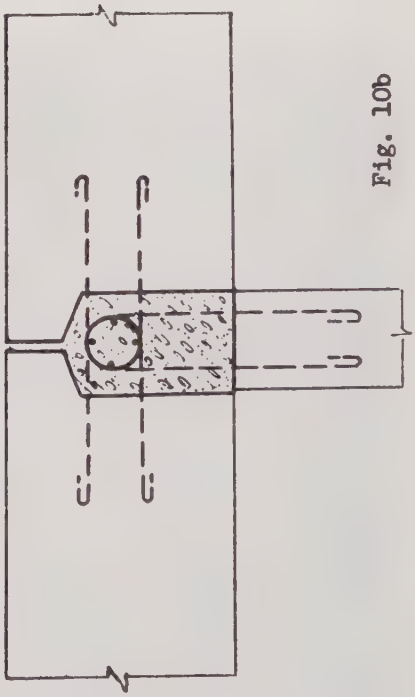


Fig. 10b

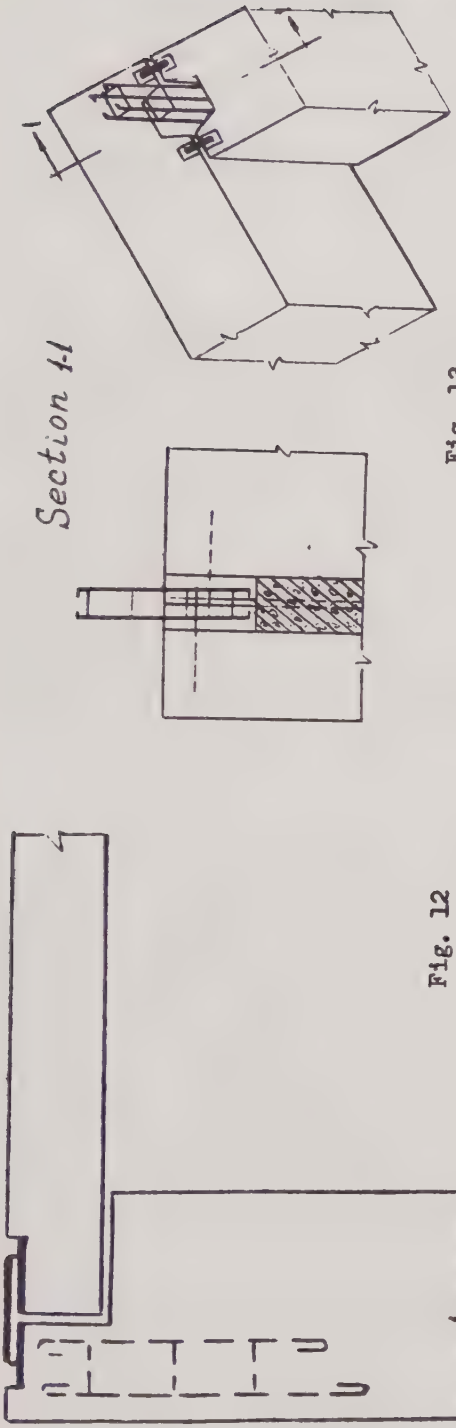


Fig. 12

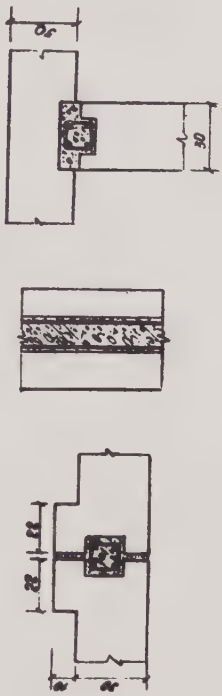


Fig. 13

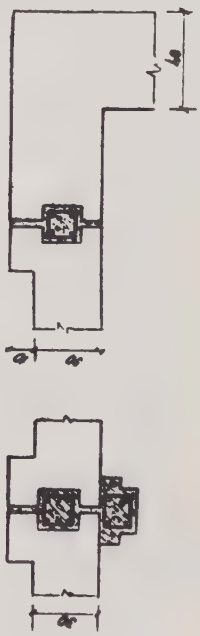


Fig. 14

DISCUSSION

K. Kubo, University of Tokyo, Japan:

I much appreciated your paper presented. Is such an interconnection device as you mentioned widely in use, and is there any example of seismic damage to that practice?

K. S. Zavriev:

The device in question is now standard in the construction of precast buildings. Tests, both static and dynamic, have been performed on full-size specimens and have proved to be satisfactory. As to date, none of the buildings so constructed have yet experienced on earthquake.

THE EFFECT OF LATERAL EARTHQUAKE ON A HIGH BUTTRESS DAM

by

Jerome M. Raphael*

INTRODUCTION

In concrete gravity dams, constructed of monolithic units of masonry making contact at the transverse construction joints, no concern has been felt for the effect of earthquake motion parallel to the dam axis on the stability and stresses in the structure. In recent years, economy of construction has forced a gradual shift from solid concrete gravity dams to buttress dams, in which the water-bearing face is supported by a series of triangular walls or buttresses. As first developed, these walls were quite thin, closely spaced, and supported laterally at intervals by struts. With increased emphasis on high dams, buttresses have become substantial free-standing plate-like structures in which the water-bearing face is merely an upstream enlargement of the buttress itself and for which designs have been projected for heights up to 750 ft. Some concern has been felt for the stability of structures of this magnitude against earthquake in a direction normal to the plane of the buttress. This paper is a preliminary analysis of the stability of high buttresses against the effects of lateral earthquake.

THE STRUCTURE AND THE PROBLEM

An idealized view of the typical structure being considered is shown in Fig. 1. The entire structure is seen to consist of a series of independent roughly triangular walls, with maximum height of 750 ft, 60 ft thick, and spaced at 130 ft centers. The enlarged upstream portion of the buttress serves as the water barrier. This type of structure is termed a massive head buttress dam, and sometimes given more descriptive terms such as a diamond head or round head buttress dam, depending on the shape of the massive head. Each buttress is constructed independently, but makes contact with the neighboring buttress by the necessity for providing some sealing detail against the passage of water after the contraction joints open when the dam has cooled to final stable temperature. Although cement grout, asphaltic, or metallic seals are equally effective, for the design under consideration, it was assumed that the joints would be sealed with a continuous rubber cushion as shown in the drawing.

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MODE OF VIBRATION

Two types of structural action can be visualized in response to earthquake vibration normal to the plane of the buttress. In the first type, it might be presumed that each buttress will act as a free-standing structural element deflecting as a triangular cantilever about its base. This supposition was examined and discarded. Each buttress is in contact with its neighboring buttresses along the entire upstream face through the water seals or cushions. It would be impossible for all the buttresses of the dam to vibrate in unison. In the first place all the buttresses are slightly different in dimension, and hence differ in natural frequency. Thus, even if the ground everywhere had the same mode of vibration, the individual buttresses would tend to vibrate independently. Secondly, a dam of this size is of a much greater length than the wave length of the earthquake, and the ground in different parts of the dam would be moving in opposite directions. Hence any consideration of single mass vibration about the base must be set aside.

A second type of vibration may be considered which more closely approximates the physical conditions of the buttress. This would be to consider that the buttress is a triangular plate supported along two sides corresponding to the upstream face and the base of the dam, and completely free along the third side which corresponds to the downstream face of the buttress. The supported faces are neither completely free nor completely fixed. It was considered that along the upstream face, although each massive head is in contact with the neighboring massive head, the details of the water seals would be such that small shearing movements could take place, and hence small rotations of the massive head about an inclined axis parallel to the upstream face of the dam would be possible. In other words, the upstream face was considered to fulfill the conditions for simple support.

Since the base of the dam is constructed directly onto a rock foundation, it might be concluded that the base of the dam is fixed. However, even here more careful consideration of the situation shows that some slight rotation of the base can take place. When large masses of rock, such as form the foundation of a dam, are examined, it can be seen that the continuity of the rock mass is not perfect, but is interrupted generally by systems of seams and fissures. The larger the mass of rock considered in any analysis or test, the lower the resistance to deformation. Comparison of the results of tests of rock cores, bearing tests, pressure chamber tests in rock, and observation of the deflections of large dams confirm that the larger the mass of rock involved, the lower its apparent modulus of elasticity, and resistance to deformation. Hence it must be concluded that considering the large mass of the buttress dam, some foundation movement can be accommodated by opening and closing the fissures in the foundation. Secondly, even if the foundation was perfectly homogeneous, isotropic, and elastic, the support of the buttress at the base could not be considered to be completely fixed, since elastic movements of a perfectly elastic foundation would allow some rotation of the base. Considering these two factors together, it was concluded that the mathematical concept of perfect fixation could not be applied to the base of the buttress dam, and it was considered simply supported at the base.

VIBRATION OF A TRIANGULAR PLATE SIMPLY SUPPORTED ON TWO EDGES

The solution to the general problem of the vibration of a triangular plate simply supported on two edges was not found in the available literature on the mechanics of deformable bodies, and some improvisation was made from the solutions of those problems that had been attacked successfully.

In studying the vibration of a square plate with free edges, W. Ritz[1,2] found that in the second mode of vibration of the square plate, the diagonals were nodal lines, and the fundamental frequency of the plate could be determined by the equation

$$w = \frac{\alpha}{a^2}\sqrt{\frac{gD}{\gamma t}} \tag{1}$$

where

$$D = \frac{Et^3}{12(1-\mu^2)} \tag{2}$$

For the three lowest modes the values of the constant α are: $\alpha_1 = 14.10$, $\alpha_2 = 20.56$, $\alpha_3 = 23.91$. The modes of vibration corresponding to each of these fundamental frequencies are shown in Fig. 2.

Study of Fig. 2, for the second mode of vibration, shows that each quarter of the plate is analogous to the problem at hand. The outside edge of the plate is free, and the two diagonals represent lines of zero displacement but finite bending. Thus, as a first approximation, the vibration of the triangular buttress supported on two edges might be represented by the vibration of a 45° right triangle with the hypotenuse free and the two equal legs simply supported.

Circular frequency and natural period are related through

$$T = \frac{2\pi}{w} \tag{3}$$

Substituting equations (1) and (2) in (3), we obtain

$$T = \frac{2\pi a^2}{\alpha}\sqrt{\frac{12\gamma(1-\mu^2)}{gEt^2}} \tag{4}$$

or

$$T = \frac{2\pi a^2}{\alpha}\sqrt{\frac{12S(1-\mu^2)}{Et^2}} \tag{5}$$

The modulus of elasticity enters into the computation for natural period, which varies with the $\sqrt{\frac{1}{E}}$. It is difficult to predict exactly the value for E, since it varies not only with the materials and method of manufacture of the concrete, but also with age, exposure, and the manner

of loading. However, after studying a number of mass concrete mixes, the value $E = 4 \times 10^6$ psi was assumed to use in this dynamic application. In the first approximation, the 800 ft high buttress, 60 ft thick, and nearly equilateral, was approximated by a 45° right triangular plate of the same thickness and having the same area as the lateral area of the buttress shown in Fig. 3a. For this case, where a = 850 ft, the natural period was computed to be 1.92 seconds.

For a second approximation, the length of the free side of the triangle was made equal to the length of the free edge of the buttress, using equation (4) as before, but substituting a = 750 ft. Fig. 3b shows the specimen right triangle fitted into the buttress section. The free edge for this case was shorter than for the imaginary figure of the first approximation, and hence the plate was stiffer, and had a lower natural period. For an 800 ft high buttress, 60 ft thick, the natural period was computed to be 1.51 seconds.

As a third approximation, a method was sought by which the value of the characteristic parameter α could be varied according to the variation of the vertex angle. Variation of the stiffness parameter has been worked out by Cox and Klein[4] for the case of an isosceles triangular plate clamped on the two equal edges and simply supported on the third edge, varying the angle between the two clamped edges. The variation of the parameter β with the ratio h/a is shown in Fig. 4a. Although the relative rigidity of Cox and Klein's c-c-s plate is much greater than Ritz's s-s-f case, the assumption was made that the Cox and Klein parameter β and the Ritz parameter α varied similarly with h/a. Correspondence between the two cases is achieved when $\frac{h}{a} = 0.5$, the 45° right triangle. Increasing all values of α by the ratio β/β_o, where β_o is the Cox and Klein coefficient for $\frac{h}{a} = 0.5$, the second relationship for the variation of α with $\frac{h}{a}$ shown in Fig. 4 is established.

Despite the approximation involved in applying the variation in the parameter α with the variation in vertex angle when using equation (4), the published results of experiments tend to confirm its general validity. In studying the seismic behavior of hollow gravity and buttress dams, Niwa, Hatanaka, and Samukawa[5] performed a number of experiments on rubber and plastic models. Fig. 5 shows the measured deflection curve of the largest element of the rubber model of a buttress dam under lateral loading. From the inversion of the deflection curves near the supports, it can be seen that the upstream base and the foundation tend to fix the web of the buttress rather more than has been assumed in the above analysis. Thus it might be expected that the period for the model might be shorter than one computed using equation (4).

The rubber of which the model was constructed had the following properties:

Dynamic modulus of elasticity	42.9 kg/cm^2
Poisson's ratio	0.43
Specific gravity	1.27

As shown on the drawings, h and a were very nearly equal. From Fig. 4, for $\frac{h}{a} = 1$, α equals 48. For a length of free edge of 30 cm, and a thickness of 2.76 cm, the natural period computed using equation (4) is 0.21 seconds. The resonant period measured by Niwa et al was 0.180 seconds, which indicates that the relative fixation of the supported edges decreases the period by about 14 percent.

For the isosceles triangle fitted over the buttress shape shown in Fig. 3c as a third approximation, $\frac{h}{a} = \frac{500}{750} = 0.67$ and $\alpha = 28$. By making the appropriate substitutions in equation (4), the natural period for the 60 ft thick, 800 ft high buttress, was computed as 1.89 seconds.

The range of natural period found from all these approximations is not great, leading to identical response to seismic acceleration. Experiment and experience have shown that the maximum response of structures to seismic force is found for those structures having a natural period around 0.25 seconds.[6,7] For periods in excess of this critical value, the response of structures decreases in a relationship which varies inversely with the natural period. In general, analytical and experimental studies for zero damping have indicated structural response for certain periods which greatly exceed the maximum excitation. However, it has been repeatedly shown that when an allowance is made for the effects of damping, which is present in all real structures to some degree, the response is greatly reduced. Thus in studying the results of a number of such experiments, the Joint Committee[8] recommended alternative curves of response for California conditions, one to be used for design of buildings and a second for the design of other structures. Curve A of Fig. 6 represents the response of framed structures with varying amounts of damping due to partitions and curtain walls to ground accelerations of 0.10g and ranges from 0.06g to 0.02g as the natural period increases from 0.25 seconds to 0.75 seconds, according to the function $C = 0.015/T$. For structures other than buildings, which are generally simpler and with less damping, generally higher values are recommended, as shown in Curve B, in which C ranges from 0.10g to 0.03g, according to the equation $C = 0.025/T$. Accordingly, an average rounded value of 1.80 seconds was assumed as the fundamental period for an 800 ft high buttress, 60 ft thick. For buttresses of lesser height, the fundamental period was computed by letting it vary directly with the square of the height, as shown in column 2 of Table I. Applying these fundamental periods to the response spectrum of Fig. 6, the seismic factors shown in column 4 can be derived.

BUTTRESS STRESSES

The stresses in the free edge of the buttress due to lateral earthquake were found by taking a one-foot wide, 60 ft deep, beam element freely supported at the crest and at the toe of the dam. Uniform lateral load was applied to this beam element, equal to the product of the seismic factor and the weight of the beam. From this, the maximum bending stress was computed, as shown in column (6), neglecting any shear transfer from the edge element to the adjacent element. This is conservative, since any shear transfer tends to partially support the element, and reduces the computed tensile stresses. As the element vibrates under the action of seismic force, stress varies from maximum tension to maximum compression. In addition to these varying stresses, compressive stress due to

dead load is always present, with possibly some additional compression due to the live load of the reservoir. The normal state of stress due to dead load and to reservoir load varies not only with the height of the reservoir, but with the relative size of the massive head of the buttress. Accordingly, to be perfectly general, the combination of dead and water load stress was taken as equal numerically in psi to the height of concrete above the point in question in feet. This assumption forms the basis for the values in column (7).

The combined stress in column (8) is the algebraic sum of the earthquake tensile stresses in column (6) and the usual compressive structural stresses in column (7). Study of the table shows that the 60 ft thick buttress has no objectionable net tensile stresses for any height of dam. Hence it must be concluded that the free-standing buttresses should be amply stable against lateral earthquake. It can also be concluded that nothing would be gained by constructing intermediate massive lateral bracing, say at mid-height, to reduce the unsupported length of the buttresses. This would only decrease the natural period in a manner yet to be determined, and probably increase the tensile stresses

Analytical research is continuing on the fundamental response of triangular plate structures with variable conditions of support and fixation at the edges. The results of experiments published by Niwa, Hatanaka, and Samukawa represent a step toward confirmation of vibration theory for buttress dams. The next logical step will be to conduct vibration tests on full-scale dams. It is hoped that vibration machines now under construction will be used to secure these needed data.

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NOMENCLATURE

a Length of side of square or base of triangle, ft

c Clamped or fixed supports

C Seismic coefficient, fraction of acceleration of gravity

D Flexural rigidity of plate

E Modulus of elasticity, psi

f Free edge condition

g Acceleration of gravity, ft per sec^2

h Altitude of isosceles triangle, ft

s Simply supported edge condition

S Specific gravity

t Plate thickness, ft

T Natural period, sec

w Circular frequency, radians per sec

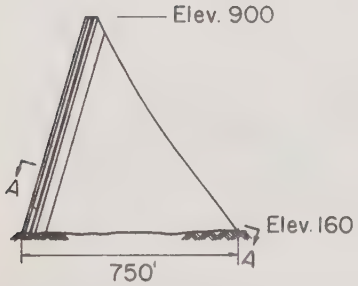
α A constant, depending on edge conditions and mode of vibration

β A constant, depending on proportion h/a

γ Density of material, pcf

μ Poisson's ratio

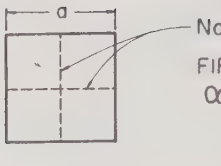
DOWNSTREAM ELEVATION



SECTIONAL ELEVATION

SECTION A-A

FIG. 1 - TYPICAL HIGH MASSIVE-HEAD BUTTRESS DAM



$$w = \frac{\alpha}{a^2}\sqrt{\frac{gD}{\gamma t}}$$

FIG. 2 - FREE VIBRATION OF SQUARE PLATES

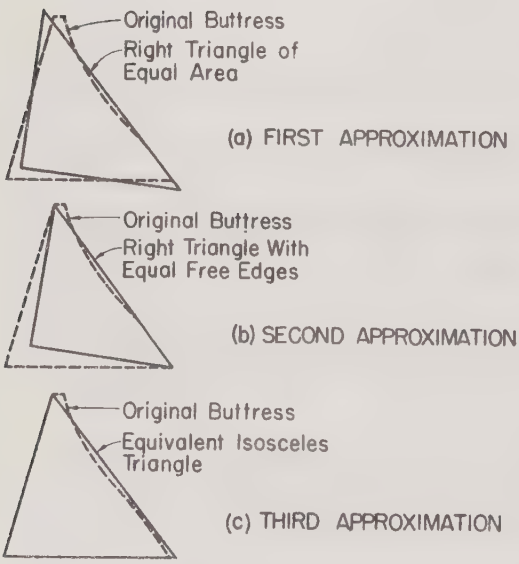
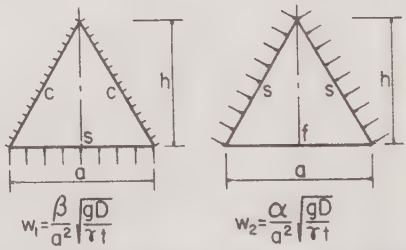


FIG. 3 - APPROXIMATIONS OF BUTTRESS SHAPE



$$w_1 = \frac{\beta}{a^2}\sqrt{\frac{gD}{\gamma t}} \qquad w_2 = \frac{\alpha}{a^2}\sqrt{\frac{gD}{\gamma t}}$$

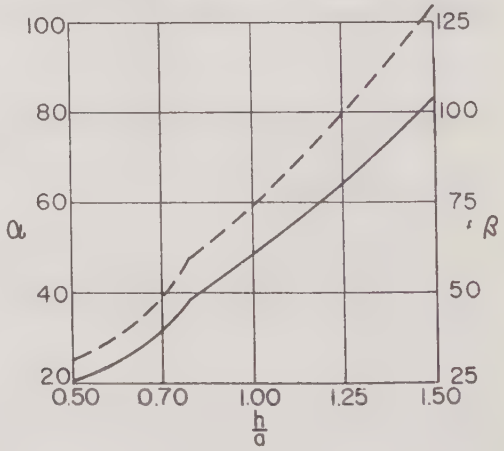


FIG. 4 - VARIATION OF α AND β WITH $\frac{h}{a}$

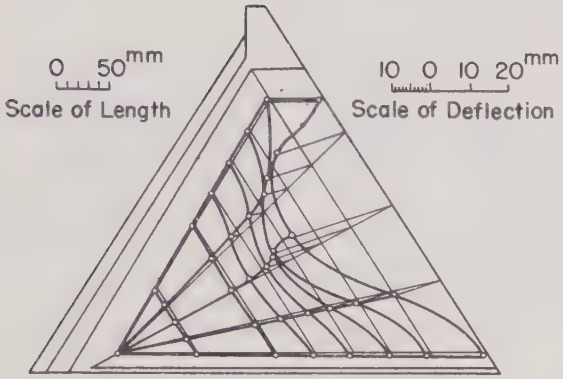


FIG. 5 - MEASURED DEFLECTION OF WEB OF RUBBER MODEL BUTTRESS

(After Niwa, Hatanaka, and Somukawa)

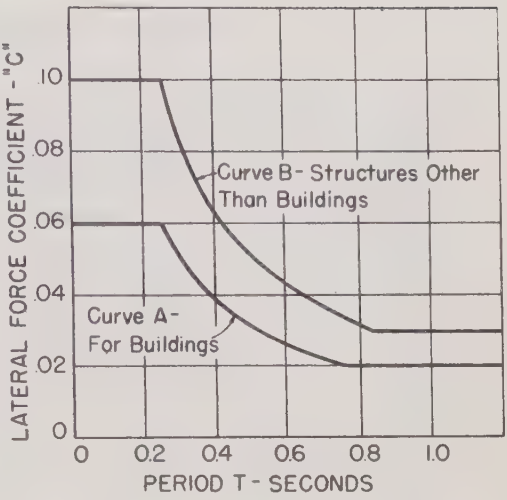


FIG. 6 - LATERAL FORCE COEFFICIENTS

TABLE I

EARTHQUAKE STRESSES IN TRIANGULAR BUTTRESS

Max Height ft	Period T sec	Seismic Factor C	Unit Load kif	Free Length ft	Bending Stress psi	Load Stress psi	Combined Stress psi
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
800	1.80	.030	.270	985	+380	-400	-20
700	1.38	.030	.270	860	+290	-350	-60
600	1.01	.030	.270	740	+214	-300	-84
500	0.70	.035	.315	615	+171	-250	-79
400	0.45	.056	.503	492	+175	-200	-25
300	0.25	.100	.900	370	+178	-150	+28
200	0.112	.100	.900	245	+ 78	-100	-22
100	0.028	.100	.900	103	+ 20	- 50	-30

DISCUSSION

M. Hatanaka, Kobe University, Japan:

Is there any record available in the United States on behaviour of dam structure and ground of dam site?

J. M. Raphael:

The U.S. Bureau of Reclamation has some comparative records of accelerations in the ground and at the crest of dams. I believe records were obtained at Hoover Dam and at Ross Dam.

SEISMIC DATA FOR THE DESIGN OF STRUCTURES

By

Jai Krishna*

Many countries are faced with the problem of establishing standards for the earthquake-resistant design without having available adequate precise instrumental data on the destructive ground motions that have occurred in the past. Despite this it is necessary to estimate the intensity of ground motion that might be expected at various locations in the future even if this requires rather drastic assumptions. It is proposed to discuss in this paper such assumptions and to present an approach for estimating possible future ground motions for which structures may be designed.

The only region where considerable number of strong motion records have been obtained is California. Elsewhere, particularly in Japan, strong motion instruments have only been recently installed in some number and there is a similar move on in India. Future shocks in these regions will perhaps make more information available regarding the ground motion and the corresponding behaviour of different structures. For the present, the data necessary to estimate the ground motion which a particular location may experience in a future shock is scanty.

With whatever data is available about the past shocks, a designer is still faced with the question as to whether at a particular site the future shocks would cause the ground motion of the same intensity as the past one did or bigger, and whether the future shocks would originate from the same faults or some other faults nearer the location of the structure. Obviously, the nearer the origin of the shock and the bigger its size, the more intense will be the ground motion. (It is only when the distance from the fault becomes very small that this general statement may not be true.) A structure designed for bigger ground motion will also cost more. A designer, therefore, has to decide upon the size of a future shock, occurring in the nearest zone of seismic activity for which the structure should be designed. In deciding this, the importance of the structure, its expected life, and probability of occurrence of the shock during the life time of the structure will have to be considered. At the same time, it will be necessary to examine how the decrease in the cost of structure by designing it for a smaller shock matches with the increased risk. There are certain structures, such as a dam or a retaining wall designed primarily to withstand horizontal forces, in which an increase in the seismic coefficient beyond a certain limit may result in an increase of their weight to such an extent that the addition to factor of safety is negligible, although the increase in cost may be considerable. Thus taking all these factors into account, the size of the design shock has to be decided upon.

There are two scales that can be used to define this shock--"magnitude" to represent the total energy released at the fault, and "intensity"

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to represent the damage that may be expected at the site. For qualitative study of the relative structural behaviour in the past shocks the "intensities" allotted to the site are of great value, but since these are based on damage reports, while damage was a function of materials, workmanship, method of construction, foundations, besides geology of the region, size and location of the shock, it is difficult to make quantitative use of this information in the design. Further, an attempt to estimate ground motion from actual failures brings in several uncertain factors regarding the structure itself so that a graph plotted between "intensity" and recorded "ground acceleration" shows a very big scatter (1). On the other hand "magnitude" by virtue of its definition is an instrumental measure of the "intensity" of ground motion at the site of the instrument and is a comparatively more precise estimate. Its definition correlates "distance", "ground motion" and total "energy" released. Although these relations are also some-what empirical yet they give the information in the form a designer needs it, and eliminates many other factors which are included in the "non-instrumental" estimation of the size of an earthquake. It is, therefore, suggested that a suitable "magnitude" of the expected shock may be chosen in the light of considerations in the previous paragraph.

Having fixed the size of the shock, it would be necessary to make certain assumptions in order to estimate the ground acceleration at the site of the structure. (At this stage it may be stated that recent studies have shown that "velocity" of the ground motion seems to have a more direct relation with damage to a structure, but, since the use of "velocity" is made in the "limit design" methods which for the present are understood well only in some simple cases, "acceleration" continues to be utilized for design purposes). The assumptions are:-

(1) The radiation of energy released from the disturbed mass in uniform in all directions, and the source of energy may be considered to be the centre of this mass from which all distances are measured.

(2) The design shock has the centre of the disturbed mass approximately at a depth of 15 miles below surface (bulk of the past shallow shocks which have caused major damage had their centre at a depth of 15 miles as an average.) For shocks of the large magnitude it is conservative to consider the release of the entire energy at the centre of the mass as the fault length may be several miles.

(3) The intensity of ground motion decreased with the square of the distance from the centre of the disturbed mass and increases with the cube root of the energy released as suggested by Gutenberg and Richter (2).

The distance-ground motion relationship has been borne out by the results of the blasts recorded by Carder and Cloud (3) and also the spectrum curves presented by Housner (4), shown as Fig. A. The relationships adopted from the work of Professors Gutenberg and Richter are:

$$\text{Log}_{10} E = 9.4 + 2.14\,M - 0.054\,M^2 \qquad (1)$$

and

$$a = C(E)^{1/3} \frac{h}{D^2 + h^2} \qquad (2)$$

Where 'M' is the "magnitude" of the shock; 'E' the energy released in ergs; 'a' the acceleration expected at a point situated at a distance 'D' from the centre of the disturbed mass, which is 'h' below ground level. The constant C included the effects of the ground conditions etc.

Comparing with the El Centro earthquake of May 18, 1940, for which h ≐ 15 miles; D ≐ 30 miles; M = 7.1; and the strong motion seismograph recorded the maximum ground acceleration equal to 0.33 g, the constant of proportionality in Eq.2 can be eliminated.

Putting the right hand side of Eq.1 equal to A, we have

$$E = 10^{A}$$

Substituting this in Eq.2, and comparing with El Centro shock mentioned above, we get.

$$a = (0.33g)\frac{h}{15}\frac{30^2+15^2}{D^2+h^2}\,10^{\frac{1}{3}(A-21.87)}$$

where the value of A for El Centro shock is 21.87.

$$\text{or,} \qquad a = \frac{24.75\,h}{D^2+h^2}\,10^{\frac{1}{3}(A-21.87)}\cdot g \qquad (3)$$

Giving different values to 'M' and 'D' and assuming h ≐ 15 miles, the values of 'a' have been worked out and shown in Fig. 1. Table 1 shows a comparison of the values thus obtained with those actually recorded in certain past California shocks.

It will be seen from Table 1 that, except for the Taft and Seattle earthquakes there is considerable agreement between the actual records with those obtained from Eq. 3 or from Fig. 1. The shocks of Taft and Seattle were perhaps produced by somewhat different type of slipping or the soil played a part in damping out waves before they reached the instruments. That is why the actual records are exceptionally low in magnitude compared with what would be expected for shocks of this size. It may be stated here that estimation of D,h and M have some margin of error and thus the estimated acceleration will represent average values that may be expected. There could be some special circumstances in which these results can be wide off the mark. For example, an instrument housed on the top of very loose soil which gets compacted quickly under vibratory force transmitted by an earthquake will record something quite different from what one would expect. The soil may exaggerate the motion due to low elastic modulus or it may filter the high frequency violent part of the shock and may record only the less violent part. Thus the geological and foundation conditions may produce exceptional results. It may, nowever, be mentioned that a study of blast tests carried out in Sweden, Canada and U.S.A. has shown that the general pattern of relationship between ground motion, distance from the blast and the quantity of charge is very similar even though the soil conditions were not the same,

and also that ground motion record and its characteristics are similar to those of an earthquake. Thus for any region unless there is specific data available, the approach suggested above for estimating the ground motion will offer reasonable guidance.

It will be noted that an effort has been made to assess the maximum ground acceleration, although it is well understood that the design of a structure is not a function of the peak acceleration only. It is necessary to take into account the duration of the shock and the variation of motion with time. In fact it is the integrated effect of the ground motion over the period of time the shock lasts, such as is given by a response spectrum analysis, that will give a true picture of the forces exerted on a structure. A study of the different earthquake records has shown that many features of the ground motion records are the same, and if their maximum ordinate and duration were the same, the overall results will be comparable. Thus it is proposed here that the shape of the record of the design shock may be assumed to be the same as that of El Centro Shock May 18, 1940, which is the strongest motion so far recorded, with the ordinates having the same ratio as the maximum values. The duration of the shock may also be the same. This approach or set of assumptions enables the designer to use the spectrum curves worked out at the California Institute of Technology, reproduced here as Fig. 2 even though they have been drawn for California shocks.

As a specific numerical example, using Fig. 2 and the multiplying factors proposed by Housner (4), we find that for a structure having a period of one second damping equal to 10 % the ordinate on the response spectrum is 3.5 ft/sec.^{2}. If the structure is located at 50 miles from the nearest active fault region, the design shock has a magnitude of 7.5 having the centre of disturbed mass 15 miles below the ground surface, and the point on the fault surface trace nearest to the structure is immediately above the centre of the disturbed mass, the maximum ground acceleration expected may be of the order of 0.20 (read from Fig. 1). Comparing with El Centro shock May 18, 1949, the multiplying factor for the spectrum will be (0.20/0.33) x 2.7 = 1.64. Thus the maximum acceleration response of a single degree of freedom structure may be 1.64 x 3.5 = 5.75 ft/sec. If the structure is water tower having a concentrated mass supported at the top of a flexible structure the entire mass may be subjected to this acceleration. On the other hand, if, the structure is more or less uniform such as a frame building, the variation of the force and shear will be as shown in Fig. 3.

The actual shape of the force diagram will be given by the following equations:-

$$\ddot{Y} = \sum_{i=1}^{N} \omega_i C_i \int_0^t a\, e^{-n_i \omega_i (t-\tau)} \sin \omega_i (t-\tau)\, d\tau$$

Where $\ddot{Y}$ is the acceleration response of the structure, $(2\pi/\omega_i)$ the period of vibration for the ith mode, 'Gi' the factor taking into account

the vibrating masses, the stiffness of the structure, and the location of point where $\ddot{Y}$ is determined; 'a' the ground acceleration, n_i the damping factor, 'N' the degrees of freedom and 't' the period of time upto the moment when $\ddot{Y}$ is determined. In actual practice, the use of this equation could be tedious and a study is being made to find approximate shapes for specific structures generally designed. Some of the building codes use the force diagram to be a triangle such that the shear diagram is parabolic.

The method suggested above will make a design shear diagram available on the assumptions that the future shock affecting the site of the structure to be designed will have general characteristics similar to those of California shocks and the behaviour of similar structures will be comparable.

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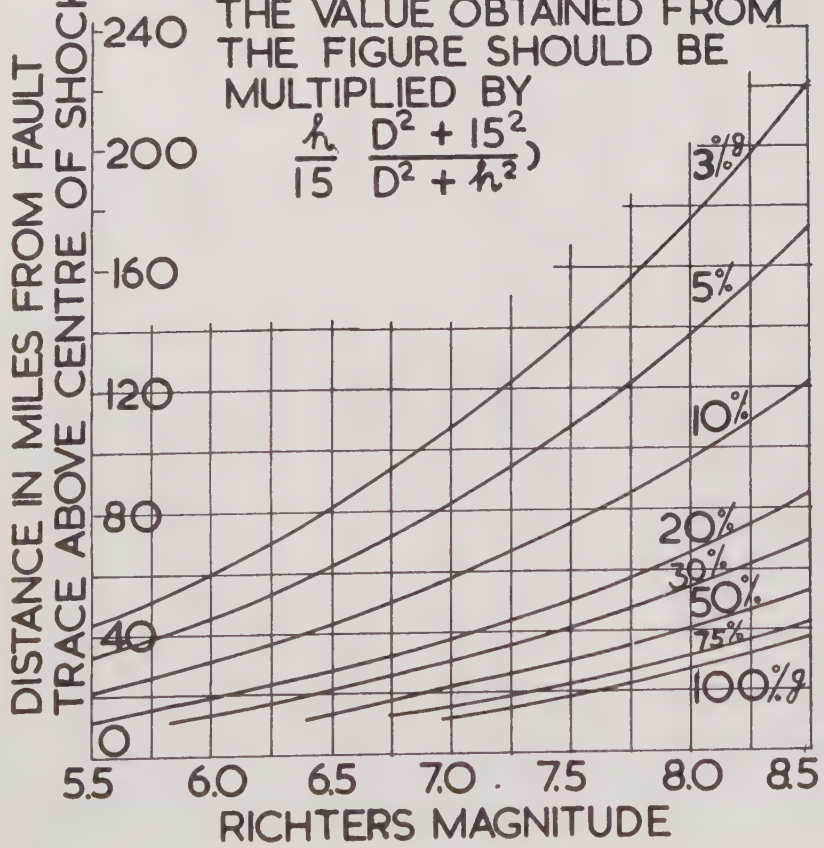
FIG.1-VARIATION OF GROUND ACCELERATION WITH DISTANCE DUE TO EARTHQUAKE SHOCKS OF DIFFERENT MAGNITUDES. (DEPTH OF CENTRE OF SHOCK ASSUMED 15 MILES BELOW GROUND LEVEL. FOR OTHER VALUES, THE VALUE OBTAINED FROM THE FIGURE SHOULD BE MULTIPLIED BY $\frac{h}{15}\left(\frac{D^2+15^2}{D^2+h^2}\right)$



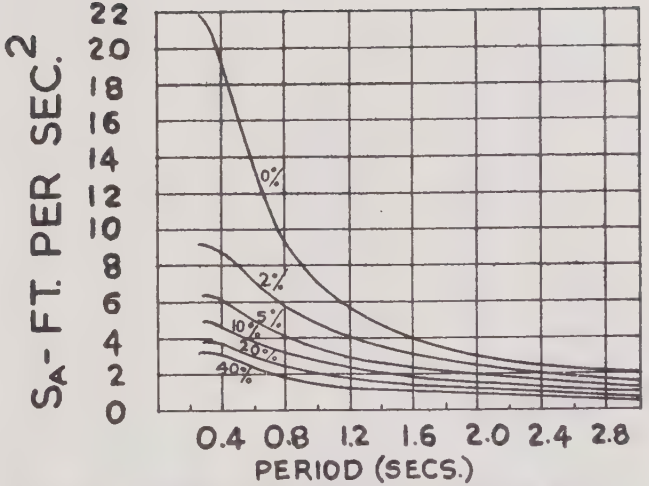


FIG.2 AVERAGE ACCELERTION SPECTRUM CURVES.

MULTIPLYING FACTORS.

1.	EI CENTRO. 18 MAY 1940	2.7
2.	EI CENTRO, 30 DEC. 1940	1.9
3.	OLYMPIA, 13 APRIL 1949	19
4.	TAFT, 21, JULY 1952	1.6
5.	VERNON, 10 MARCH 1933	1.5

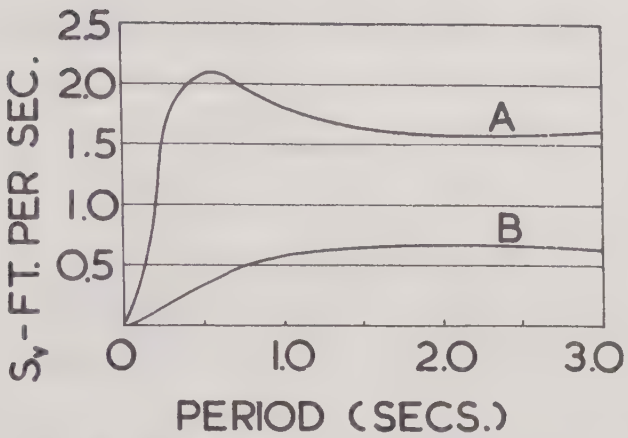
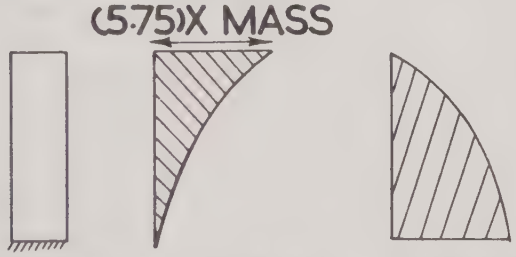


FIG. A - UNDAMPED VELOCITY SPECTRA.
A. ± 25 MILES FROM CENTRE OF LARGE SHOCK.
B ± 70 MILES FROM CENTRE OF LARGE SHOCK.



UNIFORM STRUCTURE | FORCE DIAGRAM | SHEAR DIAGRAM

FIG. 3.

Table 1

Comparison of Recorded and Calculated Accelerations

(D 15 miles)

No.	Location	D	h	M	Maximum Acc. Recorded		Acc. as calculated
		(Miles)			(Multiples of 'g') (both comp.)		
1.	El Centro, May 18, 1940	30	15	7.1	0.33	0.23	0.33
2.	El Centro Dec. 30, 1934	35	15	6.5	0.26	0.20	0.16
3.	Olympia, Wash April 13, 1949	45	45	7.1	0.31	0.18	0.28
4.	Taft, July 21, 1952	40	15	7.7	0.18	0.17	0.40
5.	Vernon, March 10, 1933	28	15	6.3	0.19	0.13	0.16
6.	St. Barbara, June 30, 1941	15	19	5.9	0.24	0.23	0.22
7.	Ferendale, Oct. 3, 1941	50	15	6.4	0.13	0.12	0.075
8.	L. A. Subway Terminal, May 10, 1933	33	15	6.25	0.065	0.04	0.11
9.	Seattle, Wash April 13, 1949	55	45	7.1	0.075	0.058	0.20
10.	Hollister, Cal March 9, 1949	10	15	5.3	0.23	0.11	0.18
11.	Helena, Oct. 31, 1935	15	25	6.0	0.16	0.14	0.19
12.	Ferendale, Feb 9, 1941	75	15	6.6	0.075	0.040	0.04
13.	Vernon, Cal. Oct. 2, 1933	17	15	5.3	0.12	0.085	0.10
14.	Ferendale, Cal. Feb 9, 1941	75	15	6.6	0.075	0.04	0.04
15.	L. A. Subway Terminal, Oct. 2, 1933	22	15	5.3	0.065	0.06	0.07

QUAKE RESISTING DESIGN OF COMPOSITE STRUCTURES IN JAPAN

by
T. Naka *
M. Wakabayashi **
S. Takada ***

Definitions

In this paper, the following expressions have the meanings hereby respectively assigned to them, that is to say:

"Composite structure" means a type of structure composed of steel skeleton clad with reinforced concrete. The strength of the structure is designed in expectation of the collaboration of them. (see Photo. 1-4)

"Steel skeleton" means steel frame composed of beam members and column members. They have not always balanced with areas or stiffenesses. "Reinforced concrete" means concrete reinforced by reinforcement of round steel bars.

Introduction

A composite structure is a structural design where the steel skeleton is encased with reinforced concrete and designed so as to work as a united body. The majority of buildings over six stories high in Japan are of this type.

This type of structure originates through the experience of Kanto Earthquake which happened on Sept. 1, 1923. The multi-storied buildings of those days were mainly built of steel skeleton encased with bricks called the "curtain wall system" and were considerably damaged from the quake due to failure of the outer encasement of bricks. Photo. 5 and 6 show examples of the damage. On the other hand, structures of steel skeleton encased with reinforced concrete designed with proper diagonal bracings and seismic resisting walls were hardly damaged at all. Photo. 7 is an example of this type of structure. Thus, the fundamantal course for seismic resisting structures has been standardized to this type of structural form currently called the composite structure.

The original sectional form of composite structural members is shown in Fig. 1 where the columns and beams were made of H-columns and I-beams. Thereafter, a built-up section of angles and plates as shown in Fig. 2 and 3 has begun to be mainly used for steel skeleton, due to the following reasons.

1. Although the strength of the encasing concrete was ignored, in early days it has been eventually taken into consideration due

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to the fact that the concrete web action has a great influence on the strength of the frame. For this reason, the open web system has begun to be mainly used for the web structures of steel skeleton columns and beams. (see Fig. 4)

2. In order to enforce the resistance of horizontal force, consideration has been made to increase the rigidity of the framework. In this connection, in order to increase the rigidity of the connection joint of the beam and column, a single gusset plate as shown in Fig. 5 has been adopted. (Explanations in detail will be made hereinafter in a latter item).

3. For the building to resist earthquakes from all directions, the column section must be designed to resist the bending moment of both principal axes.
The column sections shown in Fig.3 are examples with such conditions in consideration and the use of built-up sections deems most adequate for such cases.

4. Steel skeleton structure designed with built-up section has more flexibility in designing and also saves material.

Steel material required per 3.3m² for 7-9 story buildings as seen from the past statistics (1) is shown in Fig. 6.

As compared to the 0.3-0.4t required prior to Kanto Earthquake, it has greatly increased to 0.5-1.0t showing a great change in structural design due to the earthquake experience. Again, it has decreased to 0.3-0.6t after the 2nd. World War and the current average tonnage is about 0.4-0.5t. Furthermore, the ratio between the amount of steel skeleton and the total amount of steel (inclusive of steel skeleton and reinforcement) is shown in Fig. 7. It shows that the use of steel skeleton are decreasing and reinforcement increasing. At the present, steel skeleton occupies about 50-60% of the total amount of steel and reinforcement occupies the rest.

As described above, this type of structure is changing to reinforced concrete from the former steel frame structure and may be said to resemble reinforced concrete rather than steel frame structure.

Experimental test shows that the strength of this type of structure can be calculated with the reinforced concrete theory, assuming the steel skeleton sectional area to an equal reinforcement sectional area, and it was learned that the strength is virtually equal to reinforced concrete structure (2) (3).

The superiority of this type of structure against earthquakes is that it has greater toughness than reinforced concrete at the failure. It is due to the reason that being rigidly jointed at the connection joints of columns and beams and also the flanges of members being rigidly jointed by means of welding and rivetting, there is a great advantage of the steel skeleton structure to withstand the loads even after the concrete gives way partially. This advantage can be clearly seen during the shear failure. Experiments show that when the sectional area of flange exceeds 3% of the sectional area of the concrete, it still has enough strength even after the deformation reaches 10 times the ultimate deformation of reinforced concrete. Another advantage for earthquake resisting capacity is that, there are no slipping off nor breaking of anchorage of steel skeleton as in reinforcement.

STRUCTURAL DESIGN OF COMPOSITE STRUCTURES

Multi-storied buildings of composite structures are designed in a continuous frame. A typical example of steel skeleton and reinforced concrete frames are as shown in Fig. 8a and 8b, respectively. Steel skeleton columns are usually set on the tie beam of footing.

In Japan, the height of buildings are limited to 31 meters by the law (4) when the large open area do not exist in the surrounding; however pent-house is not included in this limit. Therfore, 9-stories is the limit above ground level but there are no limits as to the depth.

The seismic force stress is solved by assuming that the horizontal force obtained by multiplying the lateral seismic coefficient with the sum of the live load and dead load, works on each floor level. The minimum value of the lateral seismic coefficient as determined by the law is as follows.

For heights less than 16 meters: seismic coefficient 0.2
For heights more than 16 meters: seismic coefficient 0.2 plus 0.01 or more for every 4 meters
For smoke stacks, water tanks and others in this category: seismic coefficient 0.3

The seismic action causes tremendously greater bending moment than the axial force to the usual columns. On the columns adjoining a continuous quake resisting walls, a great compressive and tensile stress will act upon it, due to the wall panel working as a bar fixed to the foundation as shown in Fig. 9. And due to the action of the wall as a bar, an up and down displacement will occur at the connection joints of the wall and beam, causing a great shearing force to work on beams adjoining the wall.

STRENGTH OF COMPOSITE STRUCTURE MEMBERS

There are three methods of calculating composite structures. The first method which had been adopted during the earlier period is calculated only by the steel skeleton and reinforcement, assuming the concrete merely as a fire resisting and buckle resisting material. The second method is done in accordance with the reinforced concrete calculation theory (the straightline theory) assuming the sectional area of steel skeleton as reinforcement. The third method is calculated by the sum of each respective allowable strength of steel skeleton and reinforced concrete. The standard calculation method of The Architectual Institute of Japan (AIJ) (5) adopts the third method and uses the second method as a subsidiary method. The characteristic of this method is in calculating the strength of steel skeleton and reinforcement on seperate basis and thereby obtaining a larger allowable strength than by the calculation of reinforced concrete theory alone.

1. Section of beam (Fig. 10)

$$M_a = {}_sM + {}_rM \qquad \text{.....(1)}$$

M_a = Allowable bending moment
${}_sM$ = Allowable bending moment for steel skeleton portion $= {}_sa_t \cdot {}_sf_t \cdot {}_sj$
${}_rM$ = Allowable bending moment for reinforced concrete portion (by the straightline theory)
${}_sa_t$ = The effective sectional area of steel skeleton flange
${}_sf_t$ = Allowable tensile unit stress of steel skeleton
${}_sj$ = Distance between the gravity centre of steel skeleton flanges.

2. Section of column

When $N \leqq {}_rN_c$

$$\left.\begin{aligned} N_a &= {}_rN \\ M_a &= {}_sM + {}_rM \end{aligned}\right\} \qquad \text{.....(2)}$$

When $N > {}_rN_c$

$$\left.\begin{aligned} N_a &= {}_sN + {}_rN \\ M_a &= {}_sM \end{aligned}\right\} \qquad \text{.....(3)}$$

N = Working axial compression
N_a, M_a = Allowable axial compression and allowable bending moment, respectively
${}_sN, {}_sM$ = Allowable axial compression and allowable bending moment for steel skeleton portion
${}_rN, {}_rM$ = Allowable axial compression and allowable bending moment for reinforced concrete portion (by the straightline theory)
${}_rN_c$ = Allowable axial compression when the reinforced concrete portion is subject to axial compression

For the calculation of reinforced concrete, the allowable compressive unit stress shall be decreased as indicated below, for the reason that when the steel skeleton increases, the circulation of the concrete will be deterred therefore causing a decrease in the effective sectional area.

$$f_c' = f_c(1 - 15 \cdot {}_sP_c) \qquad \text{.....(4)}$$

f_c = Allowable compressive unit stress for concrete
${}_sP_c$ = The Ratio of steel area of the skeleton subjected to compressive stress to the gross sectional area of the column

The above equation has been formed in accordance with each respective characteristic of the steel skeleton portion and reinforced concrete portion; the former mainly carrying the bending and the latter the axial compression force. The stress of columns due to earthquakes being mainly bending, it seems effective to enlarge the steel skeleton for quake resisting purpose.

The allowable strength of equation (1)-(3) nears the ultimate strength when steel skeleton increases in comparison to reinforcement and nears the strength obtained from the straightline theory when it decreases.

3. Calculation for shear and bondage

When the nominal shearing stress $\tau = Q / \frac{7}{8} d$ of concrete exceeds the allowable shearing unit stress, the allowable shear shall be calculated as shown below.

$$Q_a = {}_sQ + {}_rQ \qquad \text{.....(5)}$$

Q_a = Allowable shearing force
${}_sQ$ = Allowable shearing force for steel skeleton portion
${}_rQ$ = Allowable shearing force for reinforced concrete portion

${}_rQ$ is the shearing force calculated by the usual reinforced concrete theory with stirrups and concrete. When the web of steel skeleton is vertical tie plate type (see Fig. 4a), calculation shall be done by assuming the tie plate as stirrups and thus ${}_sQ = 0$.

The bond unit stress of the steel skeleton and reinforcement will be calculated by the following equation for working shearing force Q.

Steel skeleton . $$ {}_s\tau_b = \frac{Q \cdot \frac{{}_sa_t}{a_t} - {}_sQ}{{}_r\psi \cdot \frac{7}{8} d} \qquad \text{.....(6)}$$

Reinforcement $$ {}_r\tau_b = \frac{Q \cdot \frac{{}_ra_t}{a_t}}{{}_r\psi \cdot \frac{7}{8} d} \qquad \text{.....(7)}$$

${}_sa_t$, ${}_ra_t$ = Sectional area of steel skeleton and reinforcement, respectively, on the tensile side
a_t = Sectional area of the total steel at tensile side
d = Distance between the compressive surface and gravity centre of the total tensile steel
${}_s\psi$, ${}_r\psi$ = Perimeter of the steel skeleton and reinforcement, respectively, at the tensile side

According to an experimental test, bondage strength per unit surface area of steel skeleton is approximately 1/2 that of plain round bar (2). In this type of structure, since there are no special attachment of dowels as on the common composite beams for the prevention of slipping, there may be lack of bondage strength. On such instances, web plates or diagonal member shall be strengthened and the shearing force shall be directly conveyed by the web structures of steel skeleton. Equation (6) is derived from these conditions in consideration.

Shear failure of members may be seen in the records of the Kanto Earthquake and since it has a great influence for an earthquake resisting capacity, the allowable shearing force has been taken on a much lower value than that derived from the experimental test to avoid unfavorable shear failure of the members.

DESIGN OF CONNECTION JOINTS BETWEEN BEAMS AND COLUMNS

Since the connection of the beam and column is subject to considerable influences of bending moment from earthquake, it is very likely to cause wide cracks and also to be damaged due to partial compressive

destruction of concrete. For this reason, strong and rigid type of joints must be considered. Generally, in reinforced concrete structure, a rigid connection may be comparatively easily obtained due to its continuous structure, but in composite structures, if the rigidity of the connection of steel skeleton is weak, the concrete may collapse before the steel skeleton commands its strength capacity. For example, experimental test revealed that when concrete encases the connection joint of the steel skeleton as shown in Fig. 11 a large crack unpermissible for general use will occur before the connection joint of the steel skeleton reaches its allowable resisting capacity.

When rivets are used, the typical Japanese method of jointing steel skeleton is as indicated in Fig. 12. In such instances, all the rivets will be subjected to only the shearing stress, so that the rigidity of the connection joints may be reliable.

When the excessive stresses occur at the extreme fibre of the gusset plate, a top angle may be attached (see Fig. 13). However, since tensile stress works on the rivet, it is used as a subsidiary material.

Recently, in order to increase the rigidity of the connection joints, welding is occasionally used. Fig. 14 shows a typical welding joint. The flange of beam is butt welded to the column.

Fig. 15 shows another instance where the columns are connected to the continuous beams. It was designed as to easing the designing condition of welding joints from the view point that the tensile stress of the flange of the steel skeleton occuring from the bending moment will be reduced by the axial compression which works on the column.

Care must be taken at the connection joint of beams and columns as it is subject to bending moment from seismic force as shown in Fig. 16. In such inctances, as the shaded portion which is a common portion of the beam and column is subject to shear, the concrete at this portion has considerable influence on the strength of the connection joint. Therefore, it is important to consider a connection which enables concrete to flow freely. For example, as is shown in Fig.14 & 15, when concrete cannot flow due to the horizontal diaphragm,it is required to place a comparatively thick web plate to resist in lieu of the concrete.

Fig. 17 shows a similar type to Fig. 14 considering the flow of concrete. Fig. 17(b) shows a vertically placed stiffener and although stress concentration occurs when the tensile stress of the beam flange is conveyed to the column, if the thickness of the column flange is equal to or larger than that of the beam flange, experimental test revealed that it is of no important matter concerning the problem of strength (6).

Bond failure and shear failure may occur due to the shear which works on this portion, but as for the latter, it is of no important matter unless the width of the concrete is extremely small. The bond failure may cause partial destruction of concrete at the compressive side of Section A-A indicated in Fig. 16 and care must be taken to avoid this. A counter measure for such instances may be done by connecting the beam flange and column with as much rivets and welding as possible. In other words, as it is a matter of concrete to flow freely and to avert the bond failure, the connecting gusset plate of

Fig. 17 may be alternated to that of Fig. 18. Experiments are recently being done on simplifying of such steel skeleton connection joints.

The allowable strength of the connection joints is calculated by the sum of the steel skeleton and reinforced concrete portion, similar to beams and columns.

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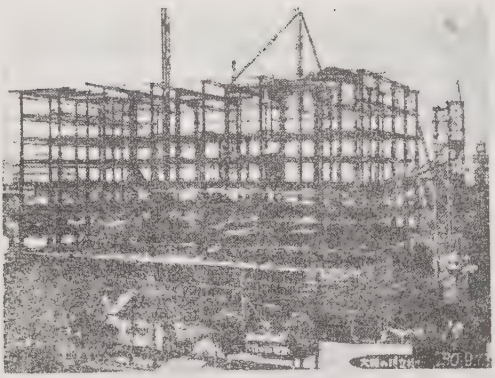
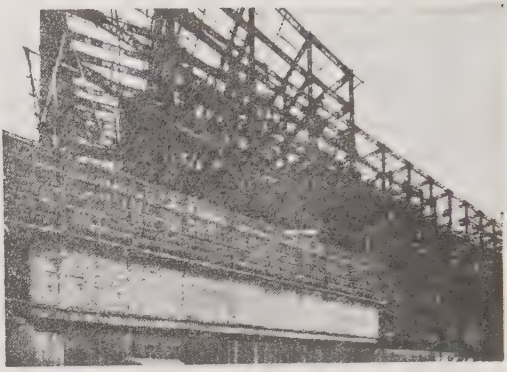


Photo. 1



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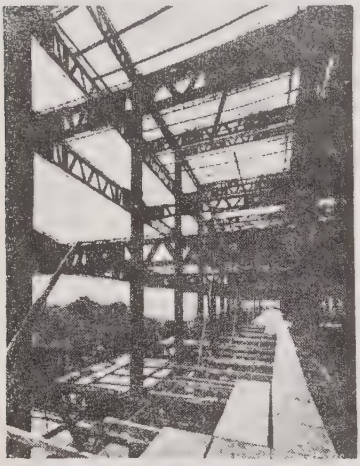


Photo. 3

Photo. 4

Photo. 5a

Photo. 5b

Photo. 6

Photo. 7

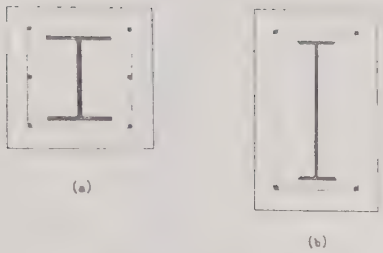


Fig. 1 Original type of sections of members
(a) Section of column
(b) Section of beam

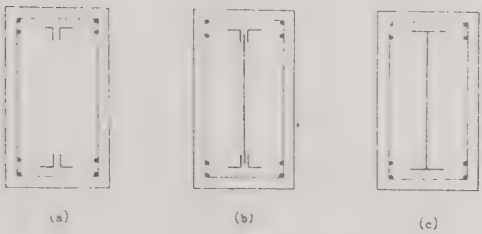


Fig. 2 Type of beam sections currently used

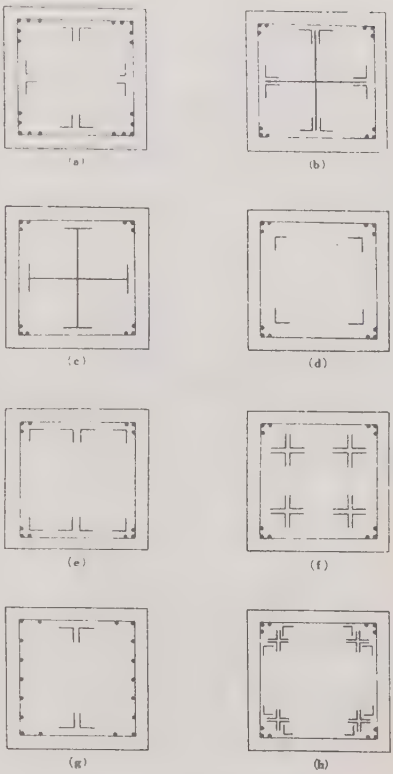
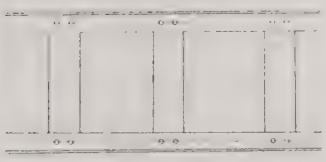
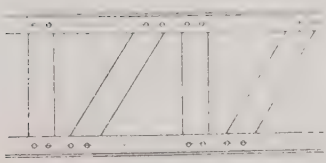


Fig. 3 Type of column sections currently Used



(a) Vertical tie plate type

(b) Warren type



(c) Pratt type

Fig. 4

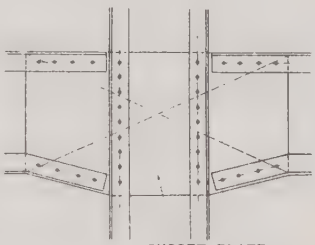


Fig. 5 Connection joint of columns and beams with a single gusset plate

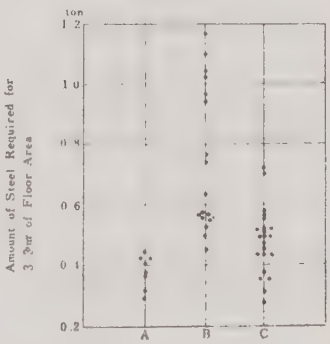


Fig. 6 A: Before Kanto-Earthquake
B: after Kanto-Earthquake
C: after 2nd War

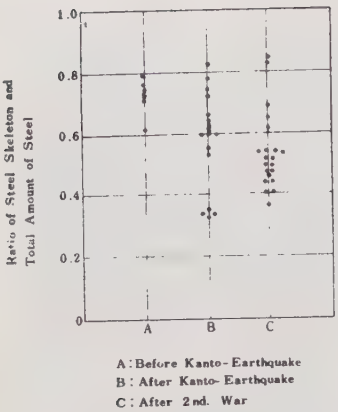


Fig. 7

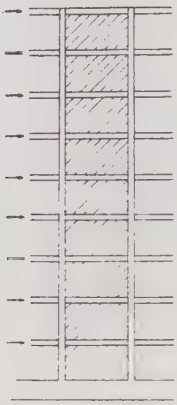


Fig. 9

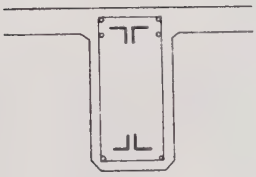
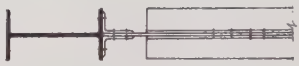


Fig. 10 Section of beam



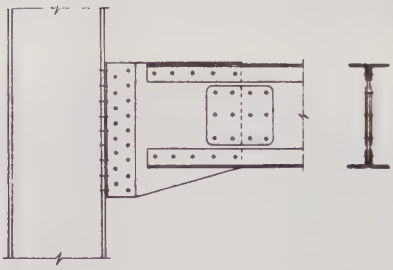
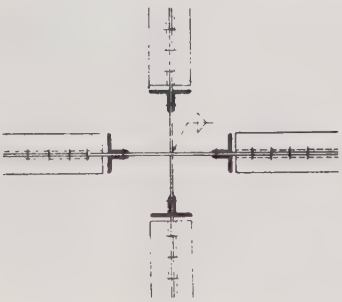


Fig. 11 Semi-rigid connection of steel skeletons (joint using clip angles)



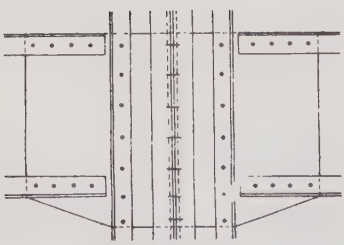


Fig. 12a Joint using gusset plate

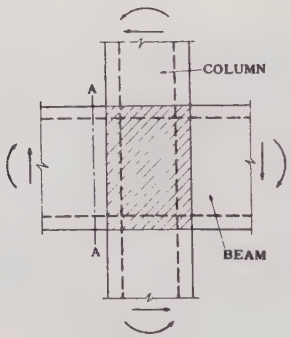


Fig. 16 Stresses at connection joints due to seismic force

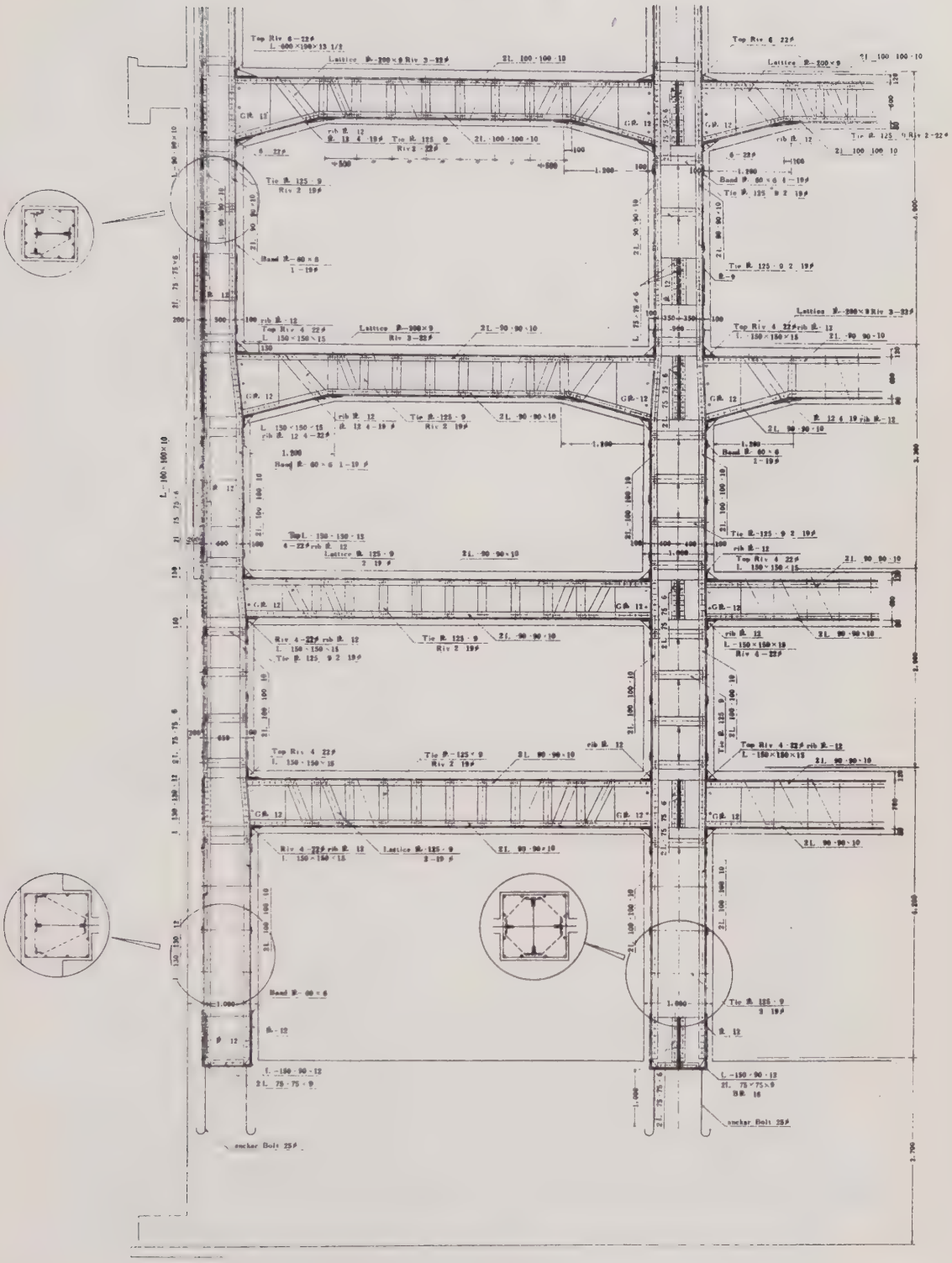


Fig. 8a Typical example of steel skeleton frame

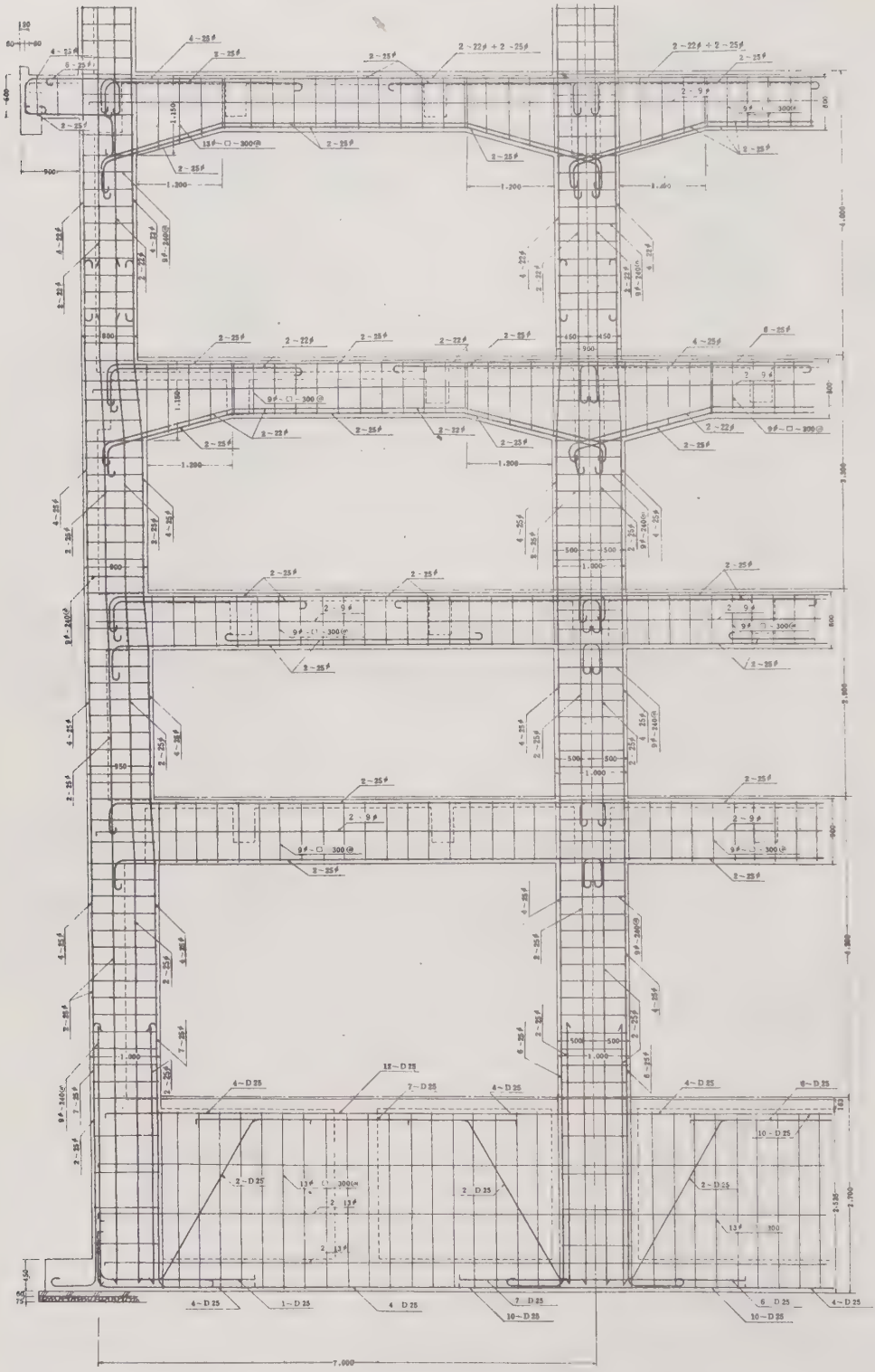
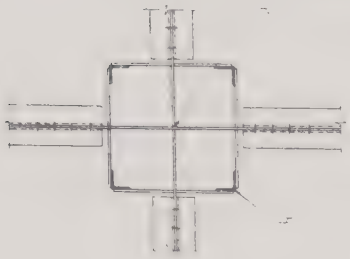


Fig. 8b Typical exmaple of reinforced concrete frame



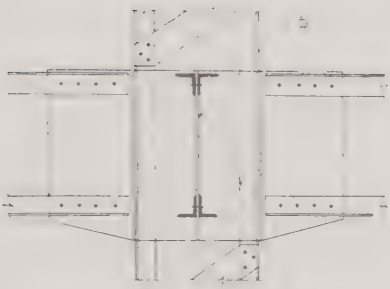
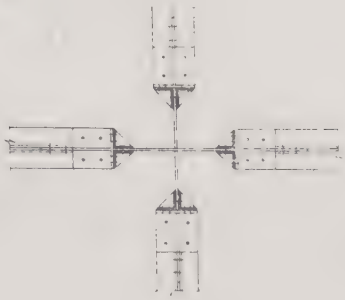


Fig. 12b Joint using gusset plate



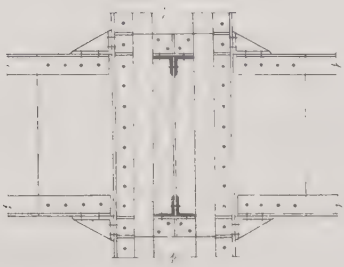
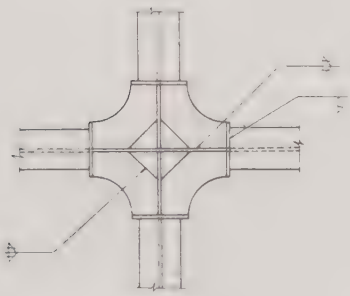


Fig. 13 Joint using gusset plate and subsidiary top angles



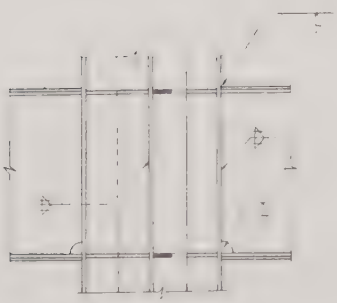
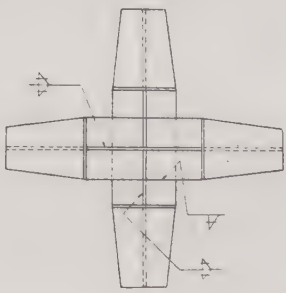


Fig. 14 Type of welding joints using horizontal diaphragms



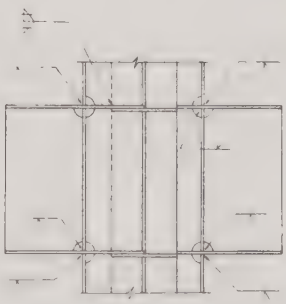
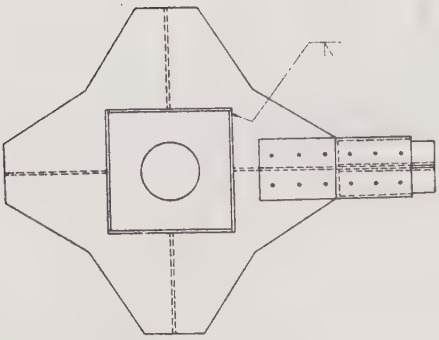


Fig. 15a Type of welding joints considering the continuity of beam



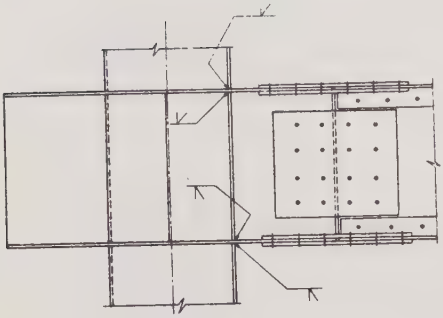
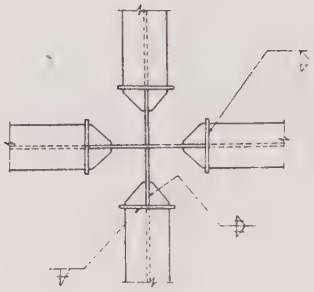


Fig. 15b Type of welding joints considering the continuity of beam



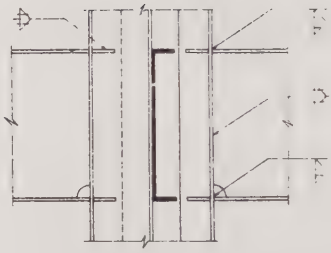
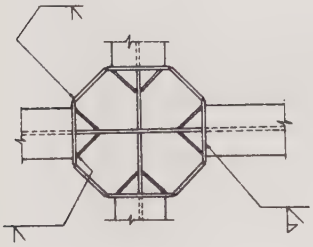
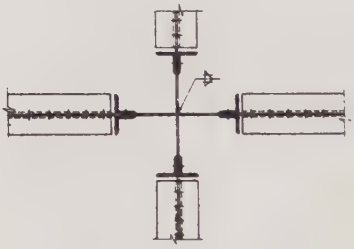


Fig. 17a Type of welding joints considering the flow of concrete





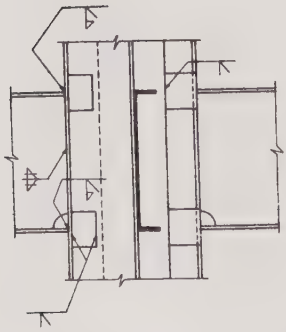


Fig. 17b Type of welding joints considering the flow of concrete

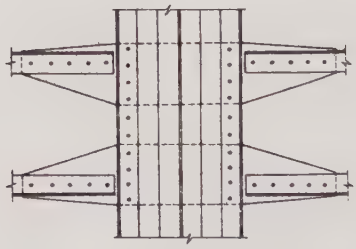


Fig. 18 Modified type of joints using gusset plate

DISCUSSION

W. P. Edwards, Edwards & Clendon Consulting Engineers. New Zealand:

In Japanese practice, plates are connected to flanges with rivets. Is there any objection to welding instead of rivets?

T. Naka, M. Wakabayashi, S. Takada:

It is more rational to connect lattice plates with flange by welding rather than by rivets, so far as the strength is concerned; chiefly from economical reason rivets are generally used.

S. Okamoto, University of Tokyo, Japan:

I appreciate your result of structural testing using the test piece of prototype scale. I would like to ask you tell me whether you recognized the scale effect of structural testing.

T. Naka, M. Wakabayashi, S. Takada:

As for the members subject to bending, compression or to the combined action or the strength of the joints of members, where any scale effect can not be recognized; it is advantageous to make a test piece in a fashion similar to the construction of a building when we rivet or weld steels or fabricate concrete members using the test piece of the prototype scale.

In case of the members subject to shearing force, the scale effect was clearly perceived: The smaller the piece is, the stronger value it showed. Therefore the scale effect must be taken into account in formulating experimental results. However beams with the depth of more than thirty centimeters showed approximately equal value or the same as that of prototype.

E. Arze, University of Concepcion, Chile:

1. How are lateral forces distributed between different members?

2. Which is the design practice for Dead Loads, to resist them with the steel frame alone or by composite action?

3. Are these buildings usually designed with shear walls, diagonal bracing or only rigid frames?

4. Is it possible to obtain figures on the weight of these buildings per unit area & comparative figures of weight of round bars per unit area for reinforced concrete buildings?

5. In these type of buildings is there any reason to prefer riveting to welding of the steel frame?

T. Naka, M. Wakabayashi, S. Takada:

1. Lateral forces are distributed according to the relative magnitude of stiffness factor of concrete section alone.

2. Members are designed assuming the composite action of the steel frame and the reinforced concrete also for dead load.

3. Most of these buildings have either shear walls or diagonal bracings.

4. Commonly used figures are as follows:

Material / Construction	Concrete (m^3/m^2)	Round bars (kg/m^2)	Steel frame (kg/m^2)
Composite(6-9 storied)	0.6	40-70	70-100
R. C. (4-5 storied)	0.6	70-100	0

5. The methods which are used in joining the steel skeleton in these buildings can be classified as follows:

(a) Riveted alone
(b) Mainly riveting with subsidiary welding
(c) Riveting except joints with welding
(d) Mostly welding and riveting is used only for joints in the field
(e) Welding only

Most buildings now employ (b) or (c), because welding is more convenient for making joints than riveting. (d) and (e) are now getting used, but those are rarely used as yet because of economical reasons.

CONSTRUCTION PRACTICES FOR MULTISTORY BUILDINGS SUBJECTED TO EARTHQUAKES

By

Adolfo Zeevaert*

INTRODUCTION

The architect and engineer are challenged by nature, to control the earthquakes on the construction of buildings. At the same time man is building higher constructions on earthquake zones. Still there are many unknown forces which the engineer must investigate. The author means by this the earthquake responses, ground motions and limits of design in relation with these observations. In the aseismic design of tall buildings there are considerations in which the building will respond in a certain way, this will cause a deformation in the structure and movements between floors. Aseismic design indicates us to avoid torsions in the building and recommends the constructor to force the architectural project to a symmetrical structure. In this kind of a building, the architect should follow closely the advise of the aseismic design-engineer and numerical computations. Attention on the architectural project and finishings should be taken into consideration.

The Tower Latino Americana, 43 story office building.

Fig. 1

The aseismic design (x) gave the pattern for a symmetrical project of the building, and the possibility of having a maximum differential movement between floors of 1/2". This displacement was taken in consideration, in the finishings, (partition walls, windows and fire proofing).

(x) Aseismic design of 40 story building in Mexico City by Dr. N. M. Newmark and Dr. Leonardo Zervaert. World conference on Earthquake Engineering 1956 Berkely, Cal.

Partition Walls: The partition walls were designed in rows which did not run into the columns, Fig. 2; also these walls were tied to the floor and lost in the ceiling, Fig. 3, to take here the displacement between floors. In this case it was recommended, to prevent a movement of a 1/2", Fig. 4.

In this particular building: Siporex slabs were used 3" by 2' and 8' long. These slabs were fixed on the floor by a channel bolted to the concrete slab of the floor structure, Fig. 5, between siporex slabs, "U" wires tied them. On the ceiling a second channel fixed the slabs together and "U" shaped steel bar holds them to the next floor, Fig. 6.

* Director of Construcciones

Windows

The windows of the building were designed to take the displacement of 1/2" between floors. This was possible, having every glass on a frame pivoting in the center; so that each frame was in the possibility to take the deformation. The setting of the glasses in the frames, gave a space large enough to take a displacement also, Fig. 7.

Fire Proofing

As an efficient fire-proof on a steel structure, there was specified by tests of a light concrete with carlita (peerlite) 2" thick. These concrete sheets were poured in place and to prevent any cracking. The fireproof was specified to be poured with a separation from the steel, Fig. 8, structure to allow and to take the movement or deformation of the structure.

Mexico City Earthquake - 28 July - 1957, 1,500 buildings Fig. 9, were reported to have damages. After an inspection of many of those, it was found that the structures were too flexible and these structures did not have serious damages, although the interiors, windows and glasses were destroyed. The loss, due to these effects, went very high, so as to have the authorities unknowing the complete estimate on this loss, Fig. 10. The author considers that many of these buildings would have suffered no damages, in the case of having a proper design of the partition walls, windows and so on following the recommendations of the aseismic design-engineer. The loss of a damaged building is higher on the architectural finishing and details than on the structure. Due to the fact that in Mexico City the foundations and super structures are about the 30% of the total cost of the building, Fig. 11.

One of the buildings, where this idea of paying attention in connection with the aseismic design engineer was taken in considerattion, is the Tower Latino Americana, Fig. 12. This building suffered no monetary loss, by the owner during this earthquake. On a thorough inspection, two points were brought out to be considered in the future.

1o.) In two places, in an office area of 300,000 square ft. were the specifications of the partition walls were not followed by the tennant, these partition walls (siporex slabs) were installed on the column lines, a small crack was observed in connection between the plaster and the finishing of the column. This indicates us that the fireproof had not the right space to allow the structure free movements, Fig. 13.

2o.) In several places it was necessary to continue the partition walls (Concrete bricks) in the elevator shaft to the next floor, and no joint was left between that wall and the beams. The author observed some diagonal cracks due to the movement of the different floors, Fig. 14.

New Ideas on Partition Walls

In a tall building or in any other construction where we cannot have a structure so rigid as the partition walls, following the right aseismic design we must have a joint that may take displacement between floors so that the decoration and finishing of the building will not suffer any damage during an earthquake. With this procedure, the success will be inevitable and there will be no loss of money, Fig. 15. If the structural design is to be of beams and slabs, we cannot have a straight line to allow the displacement between floors and it will be troublesome as pointed before, in the Tower Latino Americana.

The idea in this case is to have the joint on the floor and hang the partition walls from the ceiling. By hanging the partition walls, the author had the possibility to use prestressed slabs and precast concrete slabs (light and regular concrete) in a favorable way. The prestressed slabs are easier to handle and are stronger, Fig. 16. The floor is obliged to have a steel channel for the joint. As a side line, the author would like to mention that this has been used to solve also the problem of differential settlements, which Mexico City is suffering continuously, Fig. 17.

The earthquake in Mexico City has proved the author that it pays to study and solve these problems on tall buildings. The author has estimated that if those details of the Tower Latino Americana would not have been taken in consideration, the cracks on the partition walls would have been raised up to $ 300,000.00, the glasses on windows would be broken for an approximate value of $ 200,000.00 as well as the fireproof would be useless and it would have been in danger of fire and total collapse. The author estimates that the damages could easily have gone up to $ 1.000.000.00 on this building, of a total cost of $ 4.000,000.00. Another problem that the constructor had the necessity to take in consideration was, that when he constructed the highest building in that area in Mexico City, the people were uncertain and felt unsafe in the building. So that the tennants could easily incur into panic during an earthquake. This would cause in many cases, more casualties than the damages in the building. The panic of tennants could be started by a crack on the plaster of a partition wall, even though the structure is responding to the aseismic design perfectly. It is on our conscience to try with the architects and owners to talk them into paying attention on these finishing details.

Acknowledgment

The author expresses many thanks to Dr. Nathan Newmark and Dr. Leonardo Zeevaert for their ingenuity to solve the aseismic design problem of the Tower Latino Americana, as a contribution for progress.

References

Aseismic design of 40 - story building in Mexico City by Leonardo Zeevaert, Prof. N. M. Newmark. World Conference on Earthquake Engineering 1956. Berkely, Calif.

FIG. 1 TOWER LATINO AMERICANA

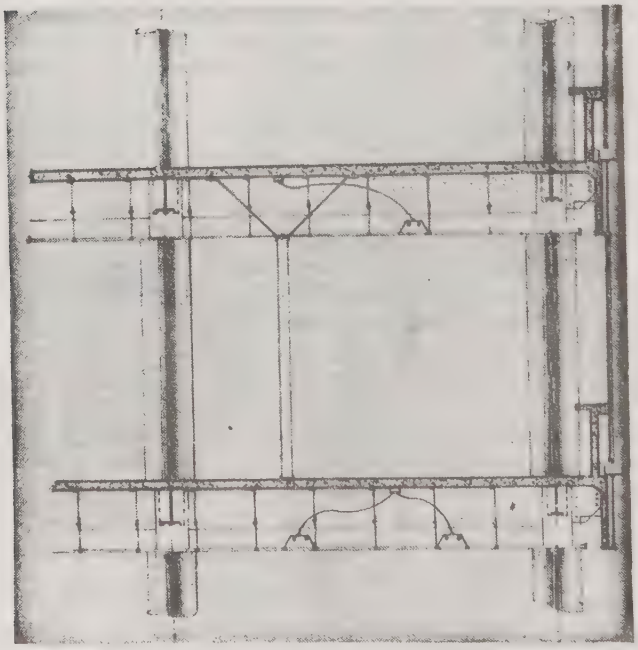


FIG. 2 DRAWING OF THE PARTITION WALLS

FIG. 3 CHANNEL TIED TO THE FLOOR.

FIG. 4 THE SIPOREX SLABS ARE TIED TO THE FLOOR WITH THE STEEL CHANNEL.

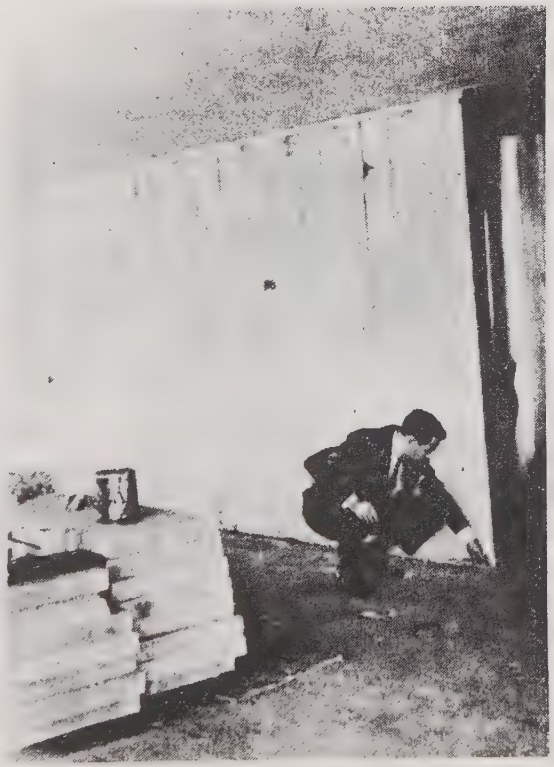


FIG. 5 A COMPLETED PARTITION WALL READY FOR FINISHING.

FIG. 6 CEILING AND PARTITION WALL.

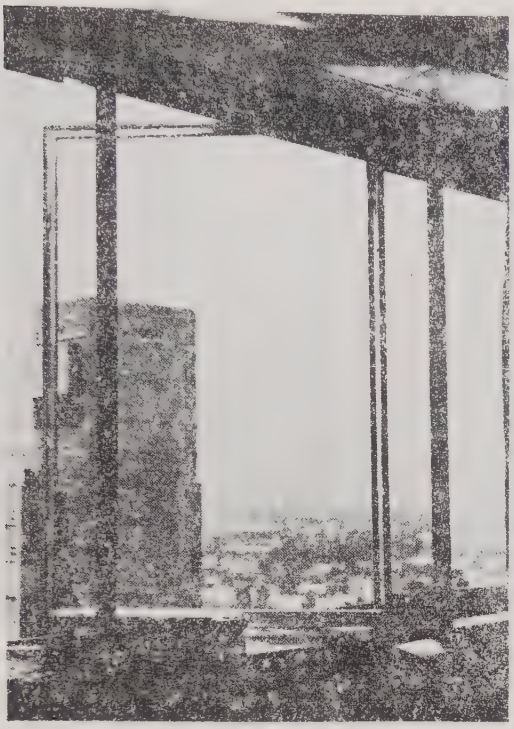


FIG. 7a WINDOWS, DESIGNED TO TAKE HORIZONTAL MOVEMENTS.

FIG. 7b WINDOWS, DESIGNED TO TAKE HORIZONTAL MOVEMENTS.

FIG. 8 TYPICAL FIRE-PROOFING OF A COLUMN.

FIG. 9a A BUILDING WITH SERIOUS DAMAGES. SUCCEDED TO CEINFORCE THE STRUCTURE, AT PRESENT IT IS IN SERVICE AGAIN.

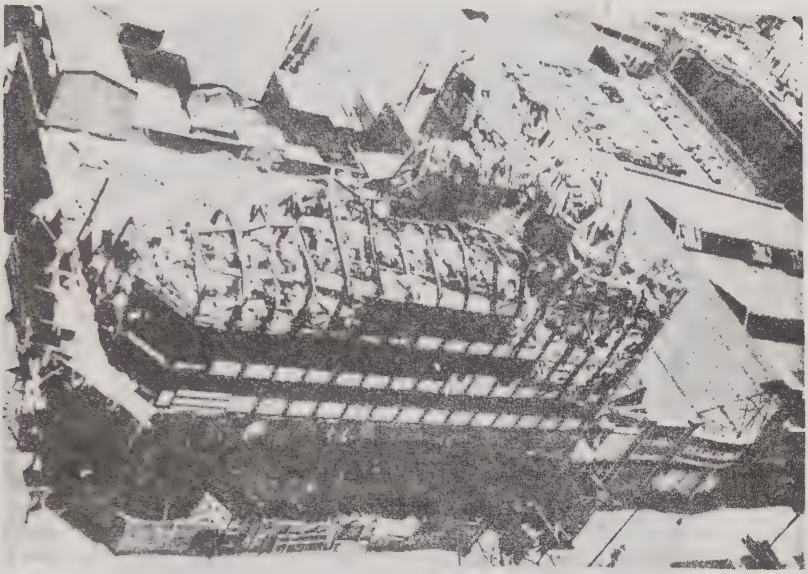


FIG. 9b A 8 STORY BUILDING UNDER CONSTRUCTION WHICH FAILED.

FIG. 9c A 7 STORY BUILDING WHICH COLAPSED.

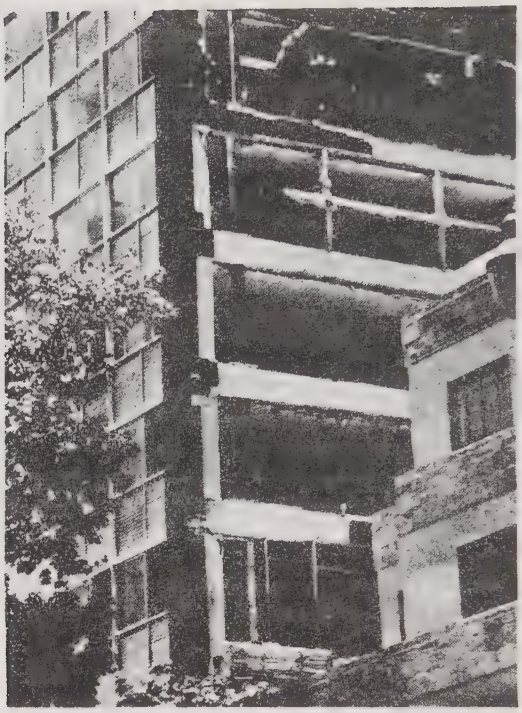


FIG. 10a A BUILDING SERIOUSLY DAMAGED ON THE CONCRETE STRUCTURE. IT SUCCEDED IN THE REINFORCEMENT IT IS IN SERVICE NOW.

FIG. 10b A BUILDING WITH STEEL STRUCTURE SERIOUSLY DAMAGED; 12 COLUMN BUCKLE. THIS BUILDING IS NO LONGER IN SERVICE.

FIG. 10c TIPICAL FAILURE OF A PARTITION WALL TIED TO THE STRUCTURE.

FIG. 10d THE POLYTECHNIC UNIVERSITY FAILED ON THE CONCRETE STRUCTURE.

FIG. 11 A BUILDING WITH STEEL STRUCTURE, UNDAMAGED. ARCHITECT. AGUSTO H. ALVAREZ CONSULTING ENGINEER: CARLOS EACALANTE M. S.

FIG. 12 TOWER LATINO AMERICANA, 43 STORIES, 182.00 METER (600 FEET). AWARD OF MERIT A. I. SC AFTER THE EARTHQUAKE 28 JULY 1957(7 DEGREE MERCALI SCALE)
CONSTRUCTED BY:
RESPONSIBLE: ADOLFO ZEEVAERT, C. E.
ARCHITECT. AGUSTO H. ALVAREZ
CONSULTING ENGINEERS:
LEONARDO ZEEVAERT Ph. D.
NATHAN M. NEWMARK Ph. D.

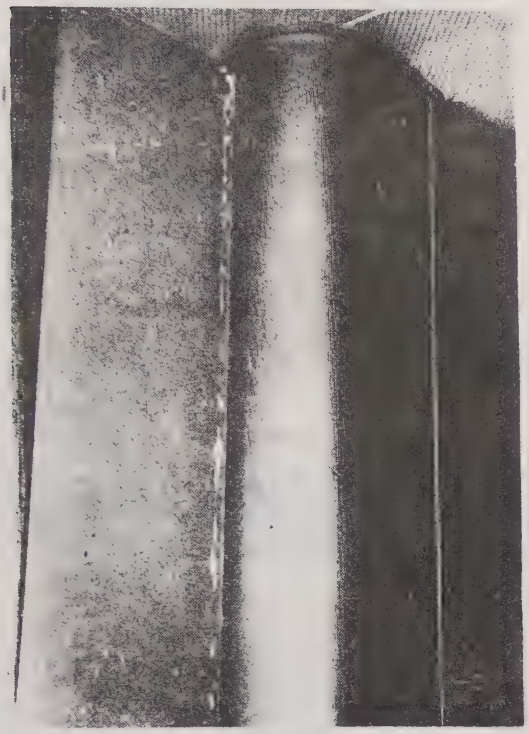


FIG. 13 A SMALL CRACK BETWEEN THE COLUMN AND A PARTITION WALL OBSERVED IN THE TOWER LATINO AMERICANA IN A PLACE WHERE SPECIFICATIONS WERE NOT OBSERVED.

FIG. 14 SMALL CRACKS IN THE ELEVATOR SHAFTS WHERE THE JOINTS BETWEEN FLOORS WERE NOT DONE PROPERLY.

FIG. 15 A NEW IDEA TO HANG THE WALLS AND FACADE TO THE CEILING THESE WALLS DO NOT HAVE A FOUNDATION, IT IS ONLY A CENTER FOOTING.

FIG. 16 PRESTRESSED SLABS FOR A FOSCADE WALL HANGING FROM THE UPPER FLOOR.

FIG. 17 JOINT BETWEEN FLOORS AND HANGING SLABS TO PREVENT DISPLACEMENTS.

BUILDING CODE PROVISIONS FOR ASEISMIC DESIGN

R. W. Binder* and W. T. Wheeler**

This paper undertakes to discuss in a very brief manner the history of earthquake codes in the United States and the development of a proposed Uniform Seismic Code for California by the Structural Engineers Association of California. Although consideration is given in this discussion to seismic development only in the United States, it must be stated at the very outset that the committee which formulated the proposed code for California was aware of the research and excellent theoretical work on earthquake engineering accomplished, and of the seismic codes established, in other parts of the world. The writers acknowledge the influence of these studies upon the deliberations of the California committee, and they are especially pleased to express appreciation of the work in the seismic field of the host country for the Second World Conference on Earthquake Engineering.

The year 1925, because of the impetus given by the Santa Barbara earthquake, may be considered as marking the real beginning of earthquake studies and research in the United States. It was in this year that, by direction of the United States Congress, the United States Coast and Geodetic Survey was given the responsibility to make investigations and reports on seismology. The work of the U. S. Coast and Geodetic Survey, in particular the publishing of strong motion earthquake records, was to have a significant influence on the development of codes.

The first edition of the Uniform Building Code of the Pacific Coast Building Officials Conference (now known as the International Conference of Building Officials) was published in 1927 and contained in the appendix a chapter on earthquake provisions, planned for optional use.

In 1928, the California State Chamber of Commerce recognized the need for a building code which would afford protection against earthquake damage through the inclusion of a section on seismic design. This project evoked wide-spread and active interest and stimulated studies and research by engineers on the subject of earthquake resistant design. The resulting requirements for the design of structures in seismic areas provided for greater security and sounder investment to the public than had been previously offered.

The first mandatory seismic codes used to any extent in the United States were published in 1933 following the March 10, 1933, Long Beach (Southern California) earthquake.(1) As a consequence of the Long Beach earthquake, two California State laws were passed.

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** W. T. Wheeler, Consulting Structural Engineer.

The Field Act, which placed complete control of structural design over public school construction in the Division of Architecture, California State Department of Public Works, authorized the Division of Architecture to approve or review all public school plans and specifications and to furnish general supervision of the work of construction. This law became effective as an emergency measure on April 10, 1933. The Division of Architecture at the same time adopted Rules and Regulations, including requirements for seismic analysis. The excellent leadership of the Division of Architecture was an influential factor in stimulating a genuine interest in seismic design and in a better understanding of design for seismic loading.

The second law, the Riley Act, which became effective May 26, 1933, made provision for design and construction to resist seismic or wind forces and was formulated for more general application than the Field Act which applied to schools only. Exempted from the Riley Act were (1) buildings located outside the limits of an incorporated area and not intended for occupancy by human beings, and (2) dwellings outside the limits of an incorporated area and planned for not more than two families. The Riley Act specified a seismic design coefficient of 2% of the design load which for most situations would mean approximately 2.5% of the design dead load. (Minor modifications of this legislation have been made from time to time.)

In addition to the State legislation, local codes were adopted in 1933 by the County of Los Angeles, the City of Los Angeles, the City of Long Beach, and other municipalities primarily in Southern California.

While there was general agreement on basic seismic matters among most California structural engineers, the development of codes after 1933 progressed along somewhat different lines in Northern and Southern California. The principal seismic codes in use following 1933 will be discussed later in the paper.

In 1957, the Seismology Committee of the Structural Engineers Association of California, at the direction of its Board of Directors, and under the chairmanship of William T. Wheeler, undertook to develop a uniform seismic code which would, in the main, resolve the important differences in the several codes used in seismic areas of the United States and particularly in California. To accomplish this objective, committee members of the Central, Northern, and Southern Associations of the Structural Engineers Association of California worked for two years, pooling their knowledge and experience in seismic matters in order to develop a uniform code.

The Committee in its early deliberations soon realized that, because of the many unknowns involved, a seismic code should be concerned primarily with establishing certain minimum ground rules and general provisions. It was expected that these would provide as uniform a degree of earthquake protection as possible for the dynamic response of structures which vary because of differences in soils, plan, size, height, and type of construction. It was concluded that it would be unreasonable and

economically unjustifiable to attempt to impose a design planned to achieve structures which would survive even the strongest earthquake without any damage. Therefore, the Committee adopted as one of its objectives the development of a seismic code which would confine its provisions to limiting the extent and type of property damage which endangers health and safety. It was also agreed that a commentary on the Code, known as a "Manual of Practice," should complement it, presenting items of detail to supplement and explain its broad considerations.

The Committee was cognizant of some, if not most, of the many variables which can influence the dynamic response of a structure to an earthquake ground motion. It realized that mere compliance with seismic code requirements would not of necessity produce a sound earthquake resistant design since the failure of an engineer to understand and appreciate fully the dynamic effects of earthquakes could result in an inadequate design. The Seismic Code of the Structural Engineers Association of California and the Manual of Practice cannot and do not attempt to provide the specific answers to seismic problems which vary from structure to structure. If the variables and their influences were known, structures could be designed in accordance with the principle of mechanics, and the Code would probably take a different form. The development of design cannot therefore be entirely analytical but must depend upon the designing engineer's experience, skill, and sound judgment in earthquake matters. It is expected that the responsible engineer will realize the dynamic nature of the problem, the complex distortions which a structure may undergo, and the related phenomena, all of which cannot be covered in specific detail by a building code. Therefore, the design of many seismic structures should become the responsibility of those experienced in the art of seismic design. This responsibility is properly assigned to those who are equipped with the knowledge and understanding of seismic research work, of special design techniques, and of the damage effects of earthquakes, particularly for the locality involved, which in our case is the United States. These concepts are expressed in the following excerpts from the foreword of the Manual of Practice:

"This manual is not intended to be a compendium of all knowledge on earthquake design, nor is it a handbook for the inexperienced engineer. The earthquake phenomenon is complex in its simplest manifestations and codes providing minimum design criteria are simplified of necessity. The 'Recommended Lateral Force Requirements' are not intended to be applied as a substitute for sound engineering judgment. The understanding, analyses, and good judgment of the experienced structural engineer in the field of engineering seismology must be relied upon for the sound and economical design and construction of specific structures. This despite the recognition of spectacular advances in knowledge in recent years, particularly in the theoretical understanding of the earthquake phenomenon."

In formulating the Structural Engineers Association of California Seismic Code, study groups from the three regional Associations were delegated the tasks of developing background material through consideration of all available research and earthquake damage studies, and of

making recommendations which would become the springboard for discussion of the Committee as a whole. Separate study groups were assigned the subjects of: Base Shear, Structural Frame, Shear Distribution, Diaphragms, Torsion, Overturning, Foundations, Set-Back, and Drift.

Inasmuch as the work of the study groups was carried on to a large extent by correspondence, and since there was an inter-relationship of the various topics, a liaison member for each of the three Associations--Northern, Central and Southern--was appointed to see that pertinent information was continually circulated among Committee members in order to avoid later conflict. When it was felt that the work of the study groups had progressed to the point of making specific recommendations, a meeting of the entire Committee was called in order to consider the findings of the various study groups.

The Committee was originally made up of sixteen members and was supplemented later by twenty members appointed to study groups. Many well qualified individuals were consulted from time to time, and thus specialized knowledge was given consideration in the deliberations and conclusions of the Committee.

In order that a sound and logical code might be developed, the Chairman cautioned the Committee not to be influenced by repeated statements which may have become cloaked with apparent authority and which tended towards a bias from which an expedient solution might be rationalized but without proper basis..

In undertaking to develop the Structural Engineers Association of California Code, the Committee gave consideration to the seismic portions of the following codes:

1. Uniform Code of the International Conference of Building Officials adopted in 1946. (This was the same code as adopted by the City of Los Angeles in 1943 and the California State Division of Architecture in 1953.)

2. Joint Committee of San Francisco Code, published in the 1952 Transactions of the American Society of Civil Engineers.[2]

3. City of San Francisco Code, adopted in 1954.

4. City of Los Angeles Code, adopted in 1957.[3]

The Base Shear "C" coefficients, in per cent of dead load "W", established by these codes are as follows:

1. Uniform Code:

$$C = \frac{60}{N + 4.5}$$

where N = number of stories above the story under consideration. Thus for a one story building, N = 0 and C = 13.3%; for a 13 story building, N = 12 and C = 3.65%; for a 21 story structure, N = 20 and C = approximately 2.5% which is the minimum coefficient permitted by the California Riley Act.

2. Joint Committee Code: C varies from 6% to 2%. Since this code uses dead load plus 1/4 live load, the coefficient for dead load only will vary from approximately 6.6% to 2.2%.

3. City of San Francisco Code: C varies from 7.5% to 3.5% based on dead load plus some percentage of live load; for dead load only, C would be approximately 8.2% to 3.8%.

4. City of Los Angeles:

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

where S = the total number of stories and N = number of stories above the story under consideration. For 1 to 13 stories S is used as 13. Thus, for this range,

$$C = \frac{60}{N + 4.5}$$

and C = 13.3% for 1 story structures; 3.65% for a 13 story structure as discussed under Item 1, Uniform Code. For structures over 13 stories, say 30 stories, C = 2.8%.

It may be of interest to discuss briefly the development of the City of Los Angeles Code value of

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

Prior to 1957, the City of Los Angeles had a height limit of 150 feet, with a maximum of 13 stories above the ground. In 1957, the height limit of 150 feet was removed and the Structural Engineers Association of Southern California was asked to develop and present for consideration by the City building authorities a revised seismic code for use in connection with structures of unlimited height. The Association decided to retain the original code value of

$$C = \frac{60}{N + 4.5}$$

up to 13 stories and to develop a formula for C which in the taller structures would approach triangular distribution for the shear load diagram. It should be noted that this type of formula not only sets the base shear value but also establishes the shear distribution.

Triangular distribution

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

was first suggested in the Joint Committee report. For uniform story masses and uniform story heights, the force diagram values of F_x for the loading diagram will vary uniformly from zero at the base to a maximum at the top and is a true triangle. It is noted that the formula

$$C = \frac{60}{N + 4.5}$$

is a hyperbolic function with the constant "4.5" forming one asymptotic line and the constant "60" representing the degree of curvature or general configuration of the hyperbola. For the formula

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

S is taken as 13 for structures 1 through 13 stories and in this range yields

$$C = \frac{60}{N + 4.5}$$

For structures above 13 stories, the objective of engineers of the Los Angeles City Building Department who developed the formula was to approach triangular distribution values in taller structures by using only that portion of the hyperbolic function that is essentially linear throughout the range where such values are desired. The resultant values of C achieved the original objective quite satisfactorily for structures above approximately 13 stories.

To this point we have discussed the base shear seismic coefficients as used by existing United States codes which vary from 13.3% down to approximately 2.5% of dead load. Many, if not most, of the Committee members were thoroughly familiar with the long and heated debates which had taken place over the years as to the magnitude of the seismic factor to be used and the degree of damage to be guarded against. It was known that the magnitude advocated by some engineers often varied with their degree of understanding of the problems involved. In addition, the use of the dynamic approach was questioned by those who advocated one seismic coefficient for all structures.

The Committee realized that the lateral forces, arrived at from the base shear coefficients specified by a code, do not necessarily represent the actual inertia forces which might be acting during an earthquake. It was also understood, from elastic analyses that have been made, that for a strong earthquake the stresses may be of greater magnitude than those indicated by the code requirements. However, it was concluded by the Committee that, from the study of earthquake damage

in the United States and from other findings, the range of coefficients in existing codes was realistic and in accord with the objectives set by the Committee.

On the basis of damage studies and the continuing research on the spectral response of resonators,[4] it was agreed that the time period of the responding structure is a significantly important parameter in determining the base shear coefficient. It was concluded that formulas using the number of stories as the variable for a measure of period did not encourage development of research in the area of determining realistic periods. In electing to compute the fundamental period in terms of the building dimensions, the Committee members, even those who originally developed the empirical formula, were aware of the short-comings of this method. However, it was felt that if the Code were based on a period concept, new and better methods of determining periods of a structure could and would be developed.

It was the opinion of the Committee that base shear coefficients, predicated on a dynamic approach in its static equivalents, should include the consideration, insofar as practical, of the response performance of different types of construction along with considering the varying degree of flexibility as well as the damping, ductility, and energy absorption capacity of various types of construction. It was expected that this approach, plus the consideration of earthquake damage studies, could result in as uniform a degree of earthquake protection as possible.

The Base Shear Study Group and the Structural Frame Study Group separately had concluded that the use of a single formula for computing the coefficient "C" for all types of structures, did not give effective consideration to completely different construction categories. In considering the various categories, they recommended a modifying factor for the coefficient "C", where "C" would be increased or decreased on the basis of the structure's energy absorption capacity and ductility; credit would also be given to structures having a second line of resistance. These recommendations were studied by the Committee, and after long and penetrating discussion, it was decided to use a factor to modify "C" in order to provide a coefficient which would vary with the type of construction. In other words, while the total base shear "V", induced into the base of a structure from an earthquake ground motion, had previously been established as $V = CW$, the new proposal would establish $V = KCW$ where K represents the modifying influence of the response performance of different types of construction with varying degrees of damping, stiffness, ductility, and energy absorption.

Base Shear coefficient "C" formulas have been stated as

$$C = \frac{60}{N + 4.5}$$

for the Uniform Code and

$$C = \frac{4.6S}{N + 0.9(S-8)}$$

for the City of Los Angeles Code. It should be noted that in using these formulas base shear coefficients vary with the number of stories. For the Joint Committee Code, the formula for C is 1.5/T and for the City of San Francisco Code, the formula for C is 2.0/T. For the City of San Francisco Code, periods of 0.27 seconds and 0.57 seconds established the coefficient cut-off values of 7.5 and 3.5% respectively. It was considered that the plateau values which established maximum and minimum "C" coefficients, with a narrow range of periods for the transition, would not provide a consistent relationship to express adequately the response performance of structures varying in periods from 0.10 seconds to 2.0 seconds and over. The Base Shear Study Group, after considerable study, made several proposals for determining "C" as a function of the period "T", and the formula finally accepted by the Committee was

$$C = \frac{5}{\sqrt[3]{T}}$$

with a maximum basic cut-off coefficient of 10%.

Figure No. 1 shows the curves for four of the codes discussed, including the Structural Engineers Association of California Code. It is believed that the new Structural Engineers Association of California Code lessens or eliminates some, if not many of the objections to the other codes. Figure No. 2 shows a nomograph for the determination of Period and Coefficient "C" with

$$T = \frac{0.05H}{\sqrt{D}} \quad \text{and} \quad C = \frac{5}{\sqrt[3]{T}}$$

Mention has been made that the response of structures having varying degrees of damping, stiffness, and energy absorption would be recognized in the final evaluation of a base shear coefficient "C". This recognition, in part, is accomplished through the use of a modifying factor designated "K". In its early deliberations, the Committee considered numerous types of construction and types of structures. However, in order to reduce the complexity of the problem, it was decided to limit K values to four types of building construction which are most prevalent. It was agreed that further studies would be necessary in order to consider any other satisfactory structural framing system not presently covered by the Code.

"K" values were established for the various types of structures and bracing systems as follows:

(a) K = 1.0 is established for structures having a complete "Vertical Load Space Frame" designed to carry all vertical loads, dead and live, and with shear walls or other vertical resisting bracing systems which are designed to resist all lateral forces. The vertical load space frame is not required to be designed to resist any percentage of lateral forces; however, while vertical load space frames need not be designed as moment resisting for lateral loads, it is anticipated that many engineers will take advantage of the frame to achieve a certain

nominal amount of lateral stiffness.

(b) K = 1.33 is established for "Box Type" structures which do not have a complete vertical load space frame capable of carrying all vertical loads. Vertical and lateral loads may be carried by various combinations of load-carrying framing, bearing walls, shear walls and vertical bracing systems without a self-contained vertical load carrying frame. Stated another way, if some of the vertical loads were carried by or relied on being carried by bearing walls, the structure would not have a complete framing system for vertical loading.

Structures under (a) and (b) above are limited to 13 stories or not to exceed 160 feet. Periods for these structures may be computed from the building dimensions as previously stated,

$$T = \frac{0.05H}{\sqrt{D}}$$

The Code states that for the purpose of computing "C", the value of "T" need not be less than 0.10 seconds and the maximum basic value of C for one and two story buildings need not exceed 10%, thus KC has a maximum of 10% x 1.33 for the Box System described under (b).

(c) K = 0.80 is established for a structure having a complete Moment Resisting Space Frame capable of resisting at least 25% of the total base shear "V". Even though the frame is capable of taking 25% of the total lateral load under this category, 100% of "V" must be designed into the total structure with all of the lateral load resisting systems participating in accordance with the principle of relative rigidities. The 100% lateral force resistance may be taken by shear walls, vertical bracing or various combinations of vertical lateral load resisting systems. The Committee felt that the 25% capacity in the moment resisting space frame would, through the use of the ductile framing, become an important second line of defense in the event of an unusually strong earthquake. For the 100% lateral resistance, periods may be computed from the building dimensions, and this same period should be used for computing the 25% on Moment Resisting Space Frames; however, the design of the frame does not require consideration of rigidities other than those of the bare frame.

(d) K = 0.67 is established for structures in which the total lateral force "V" is capable of being resisted by a Moment Resisting Space Frame in which the frame has the necessary ductility. Under this system of framing, T may be used as 0.10 seconds per story provided the frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from initially resisting the lateral forces. When designing tall slender structures under this category, it is important that the engineer control the lateral deflections or drift of a story relative to its adjacent stories.

"K" values noted in paragraphs (c) and (d) apply to structures of all heights having a moment resisting space frame. However, it should

be noted that for structures over 13 stories or 160 feet a moment resisting space frame is mandatory, and the appropriate K values of 0.80 and 0.67 apply. The moment resisting space frame is required to be made of a ductile material or a ductile combination of materials. "The necessary ductility shall be considered to be provided by a steel frame with moment resisting connections or by other systems proven by tests and studies to provide equivalent energy absorption."

Figure 3 shows curves for the four values of K discussed above.

To this point consideration has been given to base shear coefficient in terms of the response of the total structure (resonator) to a random and chaotic ground motion. The Base Shear Committee prepared lateral force calculations for three types of construction: Uniform Stiffness, Uniform Drift, and Bending Type. The spectrum used was for a ground motion modified from the strongest motion recorded in the El Centro earthquake in 1940. Results were obtained from the first mode only, and base shear coefficients were determined as a function of the period of the structure. The Committee recognized the influence on the period of a structure, of mass distribution and mass used for structural and non-structural purposes. It was concluded that if the computed base shear used depends in a large measure on the natural period of a structure, some reasonable method of calculating the period must also be used. It should be noted that insofar as periods are concerned, while arbitrary methods are set forth for obtaining the period "T", the Code provides that "Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted." It is considered that the two empirical formulas for period

$$T = \frac{0.05H}{\sqrt{D}}$$

for all structures other than a complete moment resisting space frame, for which T = 0.10 seconds per story, are probably sufficiently conservative, so that when used in connection with the formula

$$C = \frac{5}{\sqrt[3]{T}}$$

minor differences in the period of the structure will not greatly influence the value of "V". In this connection it may be noted that a conservative value of T is the minimum value because under the inverse relationship between T and C, the maximum values of "C" are thus developed.

In establishing base shear coefficients which are viewed as arbitrary values, the Committee considered among other things the geological conditions in seismic areas, the soils on which the foundation of the structure is located, the rocking or yielding of the foundations, and the influence of near and distant epicenters with respect to various types of structures. The Committee recognized the importance of soils as a variable in establishing base shear coefficients. Needless to say, many earthquake damage studies, charts, and diagrams, as well as reviews

of research and theoretical work were discussed in detail before finally establishing the Code's "KC" coefficients. It remains to be acknowledged that the intangible factor of engineering judgment, of necessity, influenced the ultimate decision.

The subject of the distribution of base shear into equivalent static forces to be concentrated at each story has been discussed briefly under the subject of Base Shear. Admittedly, if the deflected position of the structure were computed at any given instant, the equivalent static forces would follow. The concept here is that the dynamic force (in the elastic range) attributed to any mass is proportional to the mass and its deflection. It was agreed that this would be a reasonable position if the period were computed from a deflected position. It is not the purpose of this paper to discuss the many proposals for approximating the distribution which were made by the Committee. Therefore, only the final Code requirements are given. In the main, Triangular Distribution,

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

as previously discussed, is specified for most structures with the following exceptions: (1) one and two story buildings shall have uniform distribution, and (2) where the height to depth ratio of a lateral force resisting system is equal to or greater than five to one, 10% of the total force "V" shall be considered as concentrated at the top story; the remaining 90% shall be distributed as provided by the formula

$$F_x = \frac{Vw_xh_x}{\sum wh}$$

Exception (2) was, in the main, planned to give consideration to structures in which the total lateral resistance is designed around the structure's center service core. Although the Committee recognized the comparative low polar moment of inertia and the possible significant accidental torsional stresses for this type of design, it was felt that exception (2) was probably reasonably sound since this type of structure will deflect primarily in bending. Under the bending type of deflection, the forces move further up the building than is indicated by the triangular distribution formula.

The Code provides that "Every building shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the building in accordance with the following formula: $V = KCW$." The general provisions of the Code provide that the lateral "force shall be assumed to come from any horizontal direction."

Having established the criteria for the response of the total structure (resonator) to an unknown earthquake ground motion recognized to be chaotic and random, the Committee next took under consideration the problems of the design details which tie a structure together in such a manner as to cause it to vibrate as a unit.

To tie a structure together in a horizontal plane, the Code provides that floors and roofs, as well as horizontal trussing systems, acting as diaphragms, shall be designed for a minimum value of 10% applied to loads tributary from that story unless a greater coefficient of KC is required by the basic seismic formula V = KCW. The minimum basic coefficient of 10% for diaphragms was predicated in part on studies which indicated that the shear coefficient for the load of each story does not necessarily decrease as rapidly as the total seismic shear coefficient at the story as given by the basic seismic formula. The Code further provides that the horizontal shear in any horizontal plane shall be distributed to the vertical resisting elements in proportion to their rigidities. Diaphragms for design purposes are also required to have sufficient strength and stiffness to redistribute effectively the shears other than those developed from the design load or masses at the story under consideration. Examples of such other shears are the accidental torsional shears, and shears in stories at set-backs and at other points where the vertical resisting elements are discontinuous. The Committee recognized that diaphragms, including equivalent bracing systems, of given strength, could have varying degrees of flexibility and that the engineer should take into account the load-deflection characteristics of a particular diaphragm system in considering his design. It was agreed that for certain structures, and particularly for tall structures, a stiff diaphragm should be required. The Diaphragm Study Group recommended that stiff diaphragms should be considered as those having load-deflection characteristics no more limber than those of commonly used diaphragms capable of sustaining, with adequate rigidity, the applied loads, vertical and lateral, to which they may be subjected. Examples of acceptable diaphragms are 2000 psi, reinforced concrete floor slabs, or all welded type of steel decks which utilize two sheets of light gage material with one in a continuous plane and for which satisfactory test results have been recorded.(5)

The distribution of horizontal shears to the vertical lateral load resisting systems in proportion to their relative rigidities presumes, among other things, that the diaphragm system is sufficiently rigid to accomplish this distribution. Experience and research has pointed out the difficulty of accurately estimating the stiffness of a number of types of lateral load resisting systems. Admittedly, it is possible by analytical methods to over-estimate the stiffness of a vertical lateral load resisting system. Thus the designer has the problem of assigning shear to the resisting systems, giving consideration to the flexibility or stiffness of the horizontal diaphragm and to the possibility of either over or under-estimating the rigidities of the vertical resisting elements. The Committee recognized that research and studies as to the deflections of diaphragms and vertical resisting elements should continue.

Previous codes in the United States have not made provision for including the effect of torsion other than for a specifically built-in eccentricity. The Torsion Study Group recognized the importance of accidental torsion, caused in part by unbalanced dead loads, live loads, openings in floors, the application of forces, and many other factors.(6) The Study Group's report stated that, in addition, there may be torsional forces

induced from floors above or below the one under consideration. Further, differences between the assumed and true stiffness of resisting walls may be considerable. To provide for these additional forces, a conservative "C" factor for diaphragms should be required. The Code, in order to allow for accidental torsion which may result from various situations, provides ". . . . . the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level."

Seismic codes prior to 1952 recognized the "overturning" problem to a varying degree, and as a result, designs for seismic overturning moments may have been handled in several different ways by designers. Following 1952, the San Francisco and Los Angeles Codes developed positive requirements for computing overturning moments.

The Committee agreed that for tall buildings the first mode is most important for overturning. The seismic coefficients and shear distribution, in part, take into account the higher modes for local shears so that the coefficients would be overly liberal for computing overturning. Accordingly, the Code introduced a modifying factor reducing the overturning moments with an increase in the period of the structure. The Code provides that the modifying factor J shall be not less than 0.33 nor more than 1.00, where

$$J = \frac{0.5}{\sqrt[3]{T^2}}$$

Thus structures with periods approximately two seconds and over will have a reduction of 2/3, with no reduction permitted for structures with a period of approximately 0.35 seconds or less. Possibly of greatest concern to the Committee was the influence of overturning moments on the foundations. Much of the background for the requirements on overturning in the Structural Engineers Association of California Code will be covered in a paper, "Design Criteria for Shear and Overturning Moment," to be presented at this conference by John E. Rinne.

The Study Group on Foundations, while submitting several reports, concluded that it did not expect to develop results that could be codified at this time. Due to the fact that earthquakes vary and that each earthquake will further vary at different site locations, more records (e.g. data for building coupling, etc.) are needed for study by the profession before any general conclusions may be drawn. The Study Group stressed that it was important for the structural engineer, in collaboration with a soils consultant, to give studied consideration to the type of foundations which would be best suited for any major structure. Also such special consideration is needed for other structures located on sites with unusual soil conditions. It was suggested that the Code might provide a nominal increase for the seismic soil bearing values on most soils but exclude those in the classification of loose sand, silt, and other loose materials on which no increase would be permitted. Deviations from these proposed arbitrary code values could be permitted on the basis of special

foundation investigations made by a recognized soils consultant. The Study Group stressed the importance of considering most carefully possible wide differences in settlements resulting from soil bearing values used for normal loading when compared to bearing values used for short time seismic loading.

Provision has been made in the Code requiring the interconnecting of piles by ties capable of carrying tension and compression, unless it can be demonstrated that equivalent restraint can be effected by other approved methods.

The Set-Back Study Group presented a report which the Committee considered dealt with the problem of set-backs in a logical and detailed manner. The complicated dynamic situations which are inherent in developing code requirements for the many probable combinations of set-backs were discussed. Because of the difficulties involved in trying to present the detailed information in code form, it was decided to defer action on this report until a later time. Meanwhile, the Committee decided to adopt the provisions of the Los Angeles City Code which are, in general, similar to the San Francisco City Code requirements. The Code as adopted stipulates that buildings having set-backs with plan dimensions 75% or more of the corresponding base plan dimensions need not be treated as set-backs. Where the tower (set-back) dimensions are less than 75% of the base dimensions, the tower portion is required to be designed as a separate structure unless the seismic coefficient at the base of the tower is greater for the tower when considered as part of the overall structure. The Manual of Practice will present the report of the Set-Back Study Group.

The Drift Study Group concluded that research was not sufficiently advanced or comprehensive to justify specific detailed requirements for the Code. It was pointed out that, over the years, drift has been a part of design left to the judgment of the designing engineer and that drift limitations should vary for types of construction and also for the special features in a given type of structure. In general, drift requires control when (a) wind causes sway and results in discomfort to occupants of a building, and (b) when an earthquake might cause brittle non-structural elements and fragile materials, such as glass, to fail and become hazardous. It was agreed that Drift provisions should be concerned primarily with lateral wind forces which are of frequent occurrence as compared to infrequent strong earthquakes. It was felt that when drift due to wind forces is kept within the comfort zone, the earthquake drift will probably be within the safety zone, except in special cases. With these conclusions accepted, the Committee did not write into the Code any specific drift requirements but did include the general statement that "Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice." It is known that engineering considerations would require the limiting of story drift for tall slender structures, particularly for lateral wind forces. In addition, for this type of structure, it would be most important to use a method of analysis which would take into account the lengthening and shortening of columns. The Committee took the position that the

Code requirement on drift is intended as a caution to the engineer to consider the magnitude of lateral deflections insofar as these deflections may be a factor in achieving a satisfactory structure over and above consideration of strength requirements alone. The Committee realized that damage control due to deflections could be handled in several different ways. For example, brittle elements may require a design with sufficient clearance to accommodate calculated deflections, and exterior wall panels and glazing can be mounted with sufficient clearance to allow for design deflections. (Of particular interest on this subject is a paper, "Drift Limitations Imposed by Glass," to be presented at this conference by J. F. Meehan.)

Of necessity, this Code. like any building code, will undoubtedly be improved as it bears the test of time. The Committee feels that further study is particularly necessary with regard to periods of buildings, torsion effects on buildings, set-backs, drift, effect of foundation conditions for building behavior during earthquakes, and behavior of non-symmetrical buildings and structures.

At the time this paper was prepared, the Code was in the process of being adopted by the City of Los Angeles and the County of Los Angeles. Approval has also been given by the Engineering Subcommittee of the International Conference of Building Officials. (See Appendix A and B.) This acceptance of the Code attests to the achievement by those who worked on the project of their objective to develop provisions which, as Prof. R. W. Clough states in his paper, "Dynamic Effects of Earthquakes,"[7] relate "the practical requirements of a building code to the essential features of dynamic theory."

* * *

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* * *

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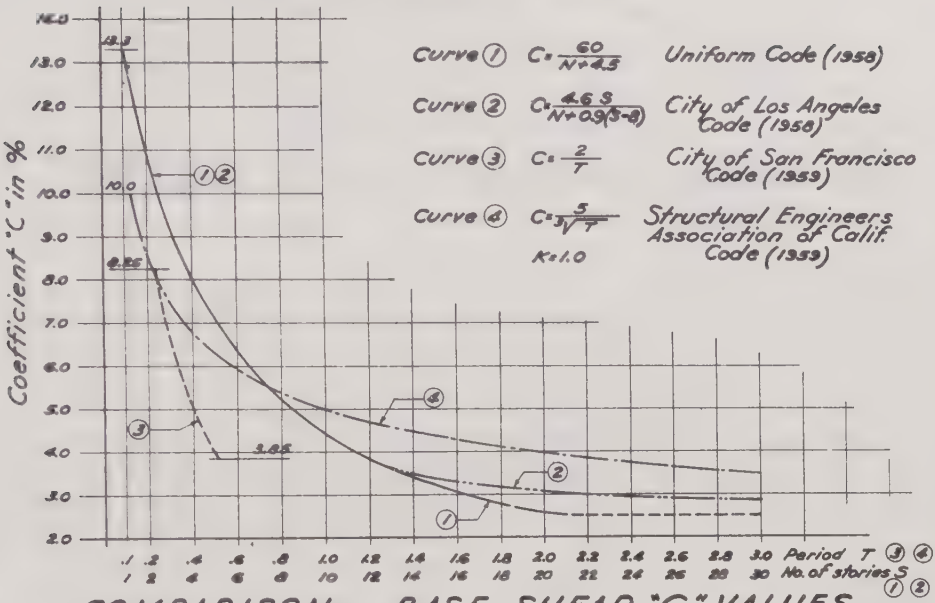
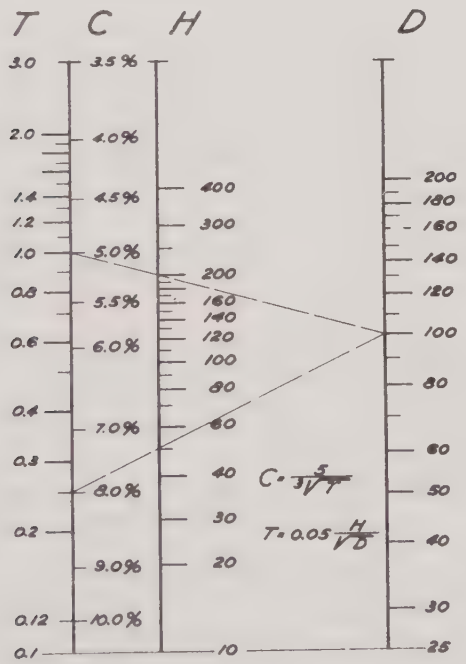

Coefficient "C" in %
Curve ① C = 60/(N+4.5) Uniform Code (1958)
Curve ② C = 4.6 S/(N+0.9(S-8)) City of Los Angeles Code (1958)
Curve ③ C = 2/T City of San Francisco Code (1959)
Curve ④ C = 5/∛T Structural Engineers Association of Calif. Code (1959)
K=1.0
Period T
No. of stories S

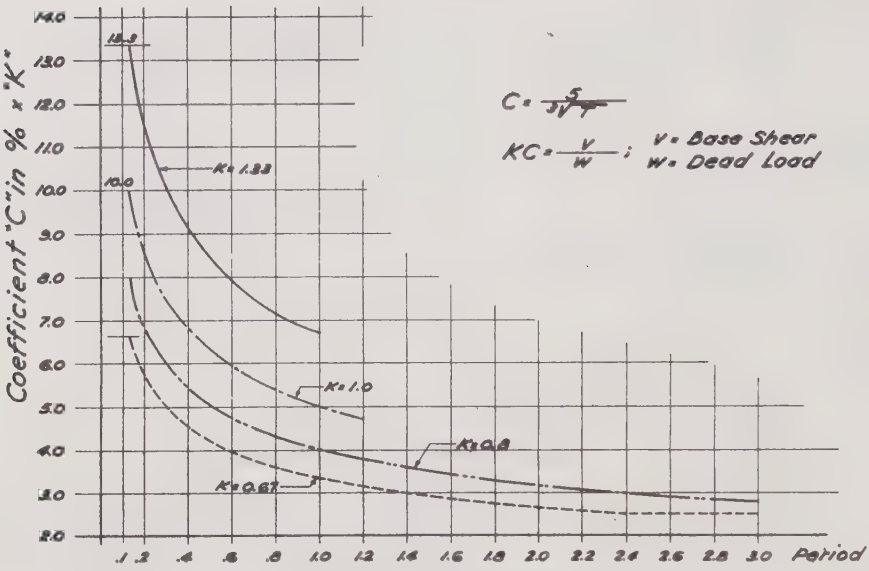
COMPARISON OF BASE SHEAR "C" VALUES

FIGURE 1



NOMOGRAPH for PERIOD "T"
and
COEFFICIENT "C"

FIGURE 2



KC VALUES
FIGURE 3

APPENDIX A

RECOMMENDED LATERAL FORCE REQUIREMENTS

SEISMOLOGY COMMITTEE

STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

Refer to Section 2312. The following provisions are suggested for inclusion in the Uniform Building Code by cities located within an area subject to earthquake shocks:

Sec. 2312. (a) General. These lateral force requirements are intended to provide minimum standards as design criteria toward making buildings and other structures earthquake-resistive. The provisions of this Section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

The provisions incorporated in this Section are general and, in specific cases, may be interpreted as to detail by rulings of the Building Official in order that the intent shall be fulfilled.

Every building or structure and every portion thereof, except Type V buildings of Group I occupancy which are less than twenty-five feet (25') in height, and minor accessory buildings, shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation. The force shall be assumed to come from any horizontal direction.

(b) Definitions. The following Definitions apply only to the provisions of this section.

SPACE FRAME is a three dimensional structural system composed of interconnected members, other than shear or bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPACE FRAME - VERTICAL LOAD-CARRYING: a space frame designed to carry all vertical loads.

SPACE FRAME - MOMENT RESISTING: a vertical load-carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments. This system may or may not be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces.

BOX SYSTEM is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces

are resisted by shear walls as hereinafter defined.

SHEAR WALL is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) Symbols and Notations. The following symbols and notations apply only to the provisions of this Section.

C	=	Numerical coefficient for base shear as defined in Section 2312(d)1.
C_p	=	Numerical coefficient as defined in Section 2312(d) 2. and as set forth in Table No. 23-D.
D	=	The dimension of the building in feet in a direction parallel to the applied forces.
F_a	=	Allowable axial stress.
f_a	=	Computed axial stress.
F_b	=	Allowable bending stress.
f_b	=	Computed bending stress.
F_p	=	Lateral forces on the part of the structure and in the direction under consideration.
F_x	=	Lateral force applied to a level designated as x.
H	=	The height of the main portion of the building in feet above the base.
h_x	=	Height in feet above the base to the level designated as x.
J	=	Numerical coefficient for base moment as defined in Section 2312(h).
K	=	Numerical coefficient as set forth in Table 23-C.
$\sum wh$	=	Summation of the product of all w_x . h_x for the building.
M	=	Overturning moment at the base of the building or structure.
N	=	Total number of stories above exterior grade.
T	=	Fundamental period of vibration of the building or structure in seconds in the direction under con-

sideration.

V = Total lateral load or shear at the base.

W = Total dead load.

EXCEPTION: W shall be equal to the total dead load plus 25 per cent of the floor live load in storage and warehouse occupancies.

W_p = The weight of a part or portion of a structure.

W_x = That portion of W which is located at or is assigned to the level designated as x.

(d) Minimum Earthquake Forces for Buildings. 1. Total lateral force and distribution of lateral force. Every building shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the building in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C = 0.10 for all one and two story buildings. T is the fundamental period of vibration of the structure in seconds in the direction considered. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T shall be determined by the following formula:

$$T = \frac{0.05H}{\sqrt{D}}$$

EXCEPTION: T = 0.10N in all buildings in which the lateral resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces.

For the purpose of computing C the value of T need not be less

than 0.10 seconds.

The total lateral force "V" shall be distributed over the height of the building in accordance with the following formula:

$$F_x = \frac{V\, w_x\, h_x}{\sum wh}$$

EXCEPTION 1: One and two story buildings shall have uniform distribution.

EXCEPTION 2: Where the height to depth ratio of a lateral force resisting system is equal to or greater than five to one, 10 per cent of the total force "V" shall be considered as concentrated at the top story. The remaining 90 per cent shall be distributed as provided for in the above formula.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. Lateral force on parts or portions of buildings or other structures. Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p\, W_p$$

The values of C_p are in Table 23-D. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. Pile foundations. Individual pile footings of every building or structure shall be so interconnected by ties each of which can carry by tension and compression a horizontal force equal to 10% of the larger pile cap loading unless it can be demonstrated that equivalent restraint can be provided by other means.

(e) Distribution of Horizontal Shear. Total shear in any horizontal plane shall be distributed to the various resisting elements in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm as well as the rigidities of the vertical resisting elements.

(f) Drift. Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) Horizontal Torsional Moments. Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. In addition, where the vertical

resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level.

(h) Overturning. Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements set forth in Section 2307, or the earthquake forces specified in this section, whichever governs.

EXCEPTION: The axial loads from earthquake force on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:

(1) The overturning moment (M) at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J \sum F_x h_x$$

where

$$J = \frac{0.5}{\sqrt[3]{T^2}}$$

The required value of J shall be not less than 0.33 nor more than 1.00.

(2) The overturning moment (M_x) at any level designated as x shall be determined in accordance with the following formula:

$$M_x = \frac{H - h_x}{H} M$$

At any level the overturning moments shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(i) Set-Backs. Buildings having set-backs wherein the plan dimension of the tower in each direction is at least 75 per cent of the corresponding plan dimension of the lower part may be considered as a uniform building without set-backs for the purpose of determining seismic forces.

For other conditions of set-backs the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

(j) Structural Frame. Buildings more than 13 stories or one hundred and sixty feet (160') in height shall have a complete moment resisting space frame capable of resisting not less than 25 per cent of the required seismic load for the structure as a whole. The frame shall be made of a ductile material or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption.

(k) Design Requirements. 1. Combined axial and bending stresses in columns forming a part of a space frame. Maximum allowable extreme fiber stress in columns at intersection of columns with floor beams or girders for combined axial and bending stresses shall be the allowable bending stress for the material used. Within the center one-half of the unsupported length of the column, the combined axial and bending stresses shall be such that

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ is equal to or less than 1.}$$

When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased as specified in Section 2302.

2. Building separations. All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

3. Minor alterations. Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

4. Unreinforced Masonry. All elements within the structure which are of masonry or concrete and which resist seismic forces or movement shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 24 and 26.

5. Combined Vertical and Horizontal Forces. In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

TABLE 23-C

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[2]

Type or Arrangement of Resisting Elements	Value of K[1]
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2312(b).	1.33
Buildings with a complete horizontal bracing system capable of resisting all lateral forces, which system includes a moment resisting space frame which, when assumed to act independently, is capable of resisting a minimum of 25% of the total required lateral force.	0.80
Buildings with a moment resisting space frame which when assumed to act independently of any other more rigid elements is capable of resisting 100% of the total required lateral forces in the frame alone.	0.67
Structures other than buildings and other than those listed in Table 23-D.	1.50

(1) The coefficients determined here are for use in the State of California and in other areas of similar earthquake activity. For areas of different activity, the coefficient may be modified by the building official upon advice of seismologists and structural engineers specializing in aseismic design.

(2) Where wind load as set forth in Section 2307 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

TABLE 23-D

HORIZONTAL FORCE FACTOR "C_p" FOR PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

PART OR PORTION OF BUILDINGS	Direction of Force	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten feet (10') in height, masonry fences over six feet (6') in height.	Normal to flat surface	0.20
Cantilever parapet and other cantilever walls, except retaining walls	Normal to flat surface	1.00
Exterior and interior ornamentations and appendages	Any direction	1.00
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smokestacks, and penthouses. Elevated tanks plus contents not supported by a building.	Any direction	0.20[1]
When resting on the ground: tank plus effective mass of its contents.	Any direction	0.10
Floors and roofs acting as diaphragms[2]	Any direction	

(1) When H/D of any building is equal to or greater than 5 to 1 increase value by 50%.

(2) Floors and roofs acting as diaphragms shall be designed for a minimum value of C_p of 10% applied to loads tributary from that story unless a greater value of C_p is required by the basic seismic formula V = KCW.

APPENDIX B

ALLOWABLE UNIT STRESSES FOR CONCRETE, STEEL AND WOOD

of the

UNIFORM BUILDING CODE of the INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS

Some of the pertinent basic allowable unit stresses for concrete, steel, and wood are tabulated below; except as noted, these stresses may be increased 33-1/3 per cent for wind or seismic forces. Wind and seismic forces for design purposes are considered separately and are not additive.

The tabulated allowable unit stresses should not be used without a thorough review of the building code and particularly the specific section of the code which applies.

Unfortunately, limitation of space precludes the inclusion of allowable unit stresses for gypsum, masonry, light gage steel, and other structural materials; of values for limit or ultimate design; of allowable unit stresses for high strength materials; and of special design techniques and applications.

ALLOWABLE BASIC CONCRETE STRESSES

General:

In determining the ratio of the modulus of elasticity of steel to concrete, the modulus for steel is 30,000,000, and for standard weight concrete the modulus is assumed equal to 1000 f'_c; where f'_c is the ultimate compressive strength of the concrete as determined by testing 6 inch diameter by 12 inch long cylinders at the age of 28 days.

Reinforcing Steel:

The most commonly used steel conforms to ASTM A15

	Intermediate Grade	Structural Grade
Ultimate tensile strength psi =	70,000 to 90,000	55,000 to 75,000
Yield point, min., psi =	40,000	33,000
Elongation in 8" min., per cent =	$\frac{1,100,000}{\text{Tensile strength}}$ (But not less than 12%)	$\frac{1,200,000}{\text{Tensile strength}}$ (But not less than 16%)
Allowable tensile working stress psi	20,000	18,000

Deformations on this steel conform to ASTM A 305.

ALLOWABLE UNIT STRESSES IN CONCRETE

DESCRIPTION		ALLOWABLE UNIT STRESSES					
		For any strgth of concrete in accordance with Sec. 2606 $n = \frac{30,000}{f'_c}$	Maximum value psi	For strength of concrete shown below			
				f'_c = 2000 psi n = 15	f'_c = 2500 psi n = 12	f'_c = 3000 psi n = 10	f'_c = 3750 psi n = 8
Flexure: fc - extreme fiber stress in:							
Compression	fc	0.45f'c		900	1125	1350	1688
Tension in plain concrete footings	fc	0.03f'c		60	75	90	113
Shear: v (as a measure of diagonal tension)							
Beams with no web reinforcement	c	0.03f'c	90	60	75	90	90
Beams with longitudinal bars and with either stirrups or properly located truss bars, but not both	v	0.08f'c	240	160	200	240	240
Beams with longitudinal bars and a combination of stirrups and truss bars (the latter bent up suitably to carry at least 0.04f'c)	v	0.12f'c	360	240	300	360	360
Flat slabs at distance d from edge of column, capital, or drop panel	Vc	0.03f'c		60	75	90	100
Footings	Vc	0.03f'c	75	60	75	75	75
Reinforced concrete shear walls		0.05f'c		100	125	150	187
Bond: u							
Deformed bars							
Top bars	u	0.07f'c	245	140	175	210	245
In 2-way footings (except top bars)	u	0.08f'c	280	160	200	240	280
All others	u	0.10f'c	350	200	250	300	350
Plain bars (must be hooked)							
Top bars	u	0.03f'c	105	60	75	90	105
In 2-way footings (except top bars)	u	0.036f'c	126	72	90	108	126
All others	u	0.045f'c	158	90	113	135	158
Bearing: fc							
On full area	fc	0.25f'c		500	625	750	938
On one-third area or less	fc	0.375f'c		750	938	1125	1405

Stresses in the above Table are used by a large number of cities in most of the Western States.

Column Formula:

Spirally reinforced columns (closely spaced spirals enclosing a circular concrete core reinforced with longitudinal steel).

$$P = A_g (0.225 f'_c + f_s P_g)$$

A_g = gross area of column

f'_c = compressive strength of the concrete (ultimate)

f_s = nominal working stress in vertical column reinforcement, to be taken at 40% of the minimum specification value of the yield point; viz., 16,000 psi for intermediate grade and 20,000 for rail or hard grade steel.

P_g = ratio of the effective cross-sectional area of the vertical reinforcement to the gross area, A_g. P_g shall not be less than 1% nor more than 8%.

Unsupported length not greater than 10 times the least lateral dimension.

Tied Columns:

The maximum permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80% of that given for spirally reinforced columns. P_g shall not be less than 1% nor more than 4%.

Long Columns:

Columns with height (h) divided by least dimension (d) ratio greater than 10 are defined as long columns. In long columns subjected to definite bending stresses, the ratio of height to width shall not exceed 20. Special designs are required for long columns.

ALLOWABLE STRUCTURAL STEEL STRESSES
in Pounds per Square Inch

Tension:

Structural steel, net section	20,000 psi
Rivets, on area based on nominal diameter	20,000 psi
Bolts, on nominal area at root of thread	20,000 psi
Butt welds, section through throat	20,000 psi

Compression:

Columns, gross section.

For axially loaded columns with value of L/r not greater than 120 $17,000 - 0.485\frac{L^2}{r^2}$

For axially loaded columns (Main members) with values of L/r greater than 120 $\left[\frac{18000}{1+\frac{L^2}{18000r^2}}\right]\left[1.6-\frac{L}{200r}\right]$

For axially loaded columns (bracing and other secondary members) with L/r greater than 120 $\frac{18000}{1+\frac{L^2}{18000r^2}}$

L is the unbraced length of the column and r is the corresponding radius of gyration, both in inches.

For main compression members, the ratio of L/r shall not exceed 180 and for bracing, struts and similar members 200.

Plate girder stiffeners, gross section	20,000 psi
Webs of rolled sections at toe of fillet	24,000 psi
Butt welds section through throat (crushing)	20,000 psi

Bending:

Tension on extreme fibers of rolled sections, plate girders, and built-up members,	20,000 psi
Compression on extreme fibers of rolled sections, plate girders, and built-up members.	
With $\frac{Ld}{bt}$ not in excess of 600,	20,000 psi
With $\frac{Ld}{bt}$ in excess of 600,	$\frac{12,000,000}{\frac{Ld}{bt}}$

in which L is the unsupported length and d the depth, of the members; b is the width, and t the thickness, of its compression flanges, all in inches.

Shearing:

Rivets	15,000
Pins, and turned bolts in reamed or drilled holes	15,000
Unfinished bolts	10,000
Webs of beams and plate girders, gross section	13,000
Weld Metal	
On section through throat of fillet weld, or on faying surface area of plug or slot weld	13,600
On section through throat of butt weld	13,000

(Stress in a fillet weld shall be considered as shear on the throat, for any direction of applied stress. Neither plug nor slot welds shall be assigned any values in resistance to stresses other than shear.)

Bearing:

	Double Shear	Single Shear
Rivets	40,000	32,000
Turned bolts in reamed or drilled holes	40,000	32,000
Unfinished bolts	25,000	20,000
Pins	32,000	
Contact Area		
Milled Stiffeners and Other Milled Surfaces	30,000	
Fitted Stiffeners	27,000	

Structural Steel shall conform to American Society for Testing Materials Specification A-7 which includes the following requirements:

Tensile strength, psi	60,000 to 72,000*
Yield point, min., psi	33,000
Elongation in 8 in., min. per cent	21
Elongation in 2 in., min. per cent	22

*The upper limit of 72,000 psi may be increased by 3,000 psi for material over 1-1/2 inches in thickness.

ALLOWABLE BASIC WOOD STRESSES
West Coast Practice

Douglas Fir used in vast majority of West Coast Structures: Basic Wood Stresses.

Description of type of stress	Allowable unit stresses in pounds per square inch for grade of lumber noted (West Coast Lumber Inspection Bureau, 1956, as modified for UBC) Douglas Fir Pacific Coast Region.	
	Select Structural Grade	Construction Grade
Bending and Tension Parallel to Grain	1,710 psi	1,350 psi
Compression Parallel to Grain	1,260	1,080
Compression Perpendicular to Grain	375	350
Horizontal Shear	110	110
Modulus of Elasticity	1,600,000	1,600,000

$$\text{Column Stress } P/A = \frac{.3E}{(L/D)^2} \qquad \text{(Max. } L/D = 50\text{)}$$

General:

Stresses (including fastenings) are based on "Normal" loadings - the usual live loads in buildings, etc.
Stresses may be increased
15% for snow loads
25% for 7 days loads (roof L.L. in most of California)
33-1/3% wind or earthquake (except as noted)
100% impact

Nails: (In many areas no increase is permitted for wind or earthquake.)

Nail Lateral Strengths: Driven perpendicular to grain of wood	6 penny	(2" long)	70 lbs.
	8 "	2-1/2"	100 "
	10 "	3"	120 "
	12 "	3-1/4"	130 "
	16 "	3-1/2"	160 "
	20 "	4"	190 "

Bolts:

Bolt values vary with diameter, length of bearing in main member, and angle to the grain.

Bolts in Double Shear: Typical bolt values in pounds per bolt.

Length of Bolt in Main Member	Loads Parallel to Grain 1/2"Ø	3/4"Ø	1"Ø	Loads Perpendicular to Grain 1/2"Ø	3/4"Ø	1"Ø
1-5/8"	910	1390	1860	430	540	660
3-5/8"	1160	2570	3990	920	1210	1460
5-1/2"		2600	4610		1750	2210
7-1/2"		2600	4630		1690	2820

Plywood Diaphragms:

Values of shear per foot allowed with common (1-5/8") width of joists or studs, for wind or earthquake.

Nails provided at all edges - blocking required at joints.

Plywood panel joints parallel to the diaphragm width shall be staggered.

		Nails 6" o.c.	4" o.c.	3" o.c.
5/16" Plywood	6 penny nails	250#/ft.	375#/ft.	420#/ft.
3/8" Plywood	8 " "	360	530	600
1/2" Plywood	10 " "	425	640	730

Nail spacing along the boundaries of the diaphragm and along shear wall connections shall not exceed two-thirds of the above values.

Diagonal Sheathing Diaphragms:

1 x 6 bds. common construction, 2 - 8 penny nails per contact, 3 - 8 penny nails at ends of boards,

Shear 300 lbs/lin. ft. for wind or earthquake

Special construction - nails as calculated.

Diaphragms:

Diaphragms are limited as to size and shape by code requirements.

PORTUGUESE STUDIES

ON EARTHQUAKE RESISTANT STRUCTURES

by

J. Ferry Borges*

1. INTRODUCTION

After the earthquake which destroyed a great part of Lisbon in 1755, reconstruction needs made it necessary to study a new type of building structure which could withstand,better than the plain masonry so far used, the destructive action of seisms.

Rough model tests were carried out in the open air at Lisbon main square (1),in which a platform, which replaced the present day shaking table, was struck by numerous soldiers armed with enormous hammers. From the experience thus achieved, it was prescribed that buildings should include a wooden frame consisting of horizontal and vertical members braced with diagonals. These frames were encased in masonry, the whole being sufficiently bonded to withstand seismic actions.These provisions formed what is believed to have been the first building code on earthquake resistant structures (2).

For more than a century and a half, building construction in Portugal complied with the rules laid down after 1755. It was only some decades ago that the generalized use of reinforced concrete made it possible to solve the problem in new bases.

Aware of the importance of the problem,Portuguese engineers decided to organize a symposium commemorating the 2nd centennial of the Lisbon earthquake of 1755, in which aseismic building problems were widely discussed (3). In 1956, the Portuguese Government appointed a committee to prepare a new code on earthquake-resisting structures which was issued in 1958 (4).

By means of a close co-operation between seismologists,geologists and engineers, a seismic map of the country was prepared. On plotting this map the following parameters were taken into account: most likely location of epicentres (between Azores and the Continent), seismic history in special as regards the two last centuries (the first informations on earthquakes in the Iberian Peninsula are about 2,400 years old), geological factors, particularly those concerning the nature of soils, and additionaly a global assessement in comparison with other

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* Research Engineer, Head Structural Studies Section, Laboratório Nacional de Engenharia Civil, Lisboa, Portugal.

areas, bearing in mind the seismic actions prescribed for those areas.

A translation of the above-mentioned Portuguese Code on Earthquake Resistant Structures as well as the map of seismic areas included therein, were sent under separate cover to the Organizing Committee of this Conference.

The basic design criteria for aseismic structures are discussed in this paper which also refers how such criteria were adopted in the Portuguese Code.

Some experimental studies on seismic actions on buildings, carried out or about to be undertaken at the Laboratório Nacional de Engenharia Civil, are also described.

2. QUANTIFICATION OF SAFETY

Among the forces acting on a structure, those due to earthquakes more than any other require that the basic notions which make possible a quantification of safety be carefully considered.

Design criteria will not be thoroughly scientific in all their stages unless a quite objective definition of safety can be achieved. For that, it is indispensable to think very carefully beforehand on the aims to be attained.

Political and social consequences of structural safety (5) being deemed outside the scope of the present paper, only economic criteria such as cost and utility shall be used as bases the considerations below.

We are thus led to think that a structure should be designed so as to minimize its generalized cost (in which all possible consequences of a collapse should be taken into account) during its anticipated life.

This cost will then consist, as a rule, of different items and while some of them can be considered to assume definite values, others particularly those concerning possible consequences of a collapse are obviously uncertain. Costs due to collapses resulting from earthquakes are particularly uncertain.

Notice that it does not by any means follow that, because a magnitude is uncertain, it can be considered with sufficient accuracy as random.

Let us then, first of all, discuss which scheme of knowledge is more adequate for studying the phenomena, the decision rules to be adopted depending on the scheme of knowledge selected. Due to their particular importance two main models are considered: random and strate-

gic.

When the random model is adopted, the decision rule consists in minimizing the sum of the expected costs, each of which is the product of a cost by its probability.

When the strategic model is adopted the decision rule would consist e. g. in the minimax criterion in which it is tried to minimize the maximum value of the cost corresponding to the different possible situations.

The adoption of statistical models implies a knowledge of the probabilities of occurrence of earthquakes of different intensities. Unhappily our knowledge in this field is very limited as regards many areas of the world.

When the probability of ocurrence of earthquakes of different intensities is known (since what is required is the probability of collapse of a given type of building), this probability has to be carefully associated with the probability of the structure to undergo a collapse of a given type for a given earthquake intensity.

If the intensity of the seismic force is measured e. g. by the maximum displacements undergone by a point of the structure, the problem may be reduced to determining the probability of occurrence of displacements above a given value during the life of the structure.

In a paper presented at the V Congress of the International Association of Bridge and Structural Engineering (6) it was shown that, assuming that the expected number of earthquakes $\lambda(K)$ is related with their magnitude K by the law suggested by Gutenberg and Richter (8)and that the maximum displacements have a Gaussian distribution — so that the probability of displacements above x for a seismic intensity K is $\phi(x,K)$ —,the expected number of times I that the maximum displacement of the structure would exceed x could be calculated by the expression

$$I = \int_0^\infty \phi(x,K)\, d_K \lambda(K)$$

By integrating this expression and neglecting some minor errors, the following result is obtained: the expected number of times that the displacements of a structure will exceed x equals the expected number of earthquakes more intense than the earthquake which on average causes the same displacement x.

Hence the practical conclusion: what is required is to know the mean value of the parameter defining the reponse of the structure to seismic actions of different intensities. From this and the probability of occurrence of earthquakes of different intensities, the probability

of each seismic force during the life of the structure can be calculated.

If a strategic model is to be adopted, it is unnecessary to have so detailed an information and the decision can be based on the analysis of selected situations. Consequently decisions supplied by this model will be as a rule, less adequate. An example of application of the minimax criterion for chosing the most suitable structural type for a group of dwellings to be built in a seismic area was presented in a previous paper (5).

3. CRITERIA FOR DEFINING STRENGTH

The definition of seismic forces cannot be separated from the method of structural analysis adopted. This is possible for current static actions which can be considered as well defined forces, it being possible to determine the strength of a structure under their action by means of elastic or plastic criteria.

A preliminary discussion of the advantages of elastic and plastic criteria in current cases can serve as a guide for the particular case of seismic actions. The chief difference between these methods, as regards the definition of strength, can be summarized as follows.

The elastic method is adequate to study the structural behaviour in stages sufficiently remote from total collapse. It is thus particularly suited for studying the structural behaviour under the most likely actions. As a rule, displacements, the appearance of cracks and even early stages of rupture can be predict with satisfactory accuracy.

Plastic methods, on the other hand, make it possible to predict with accuracy the force inducing total collapse. They are thus more adequate for studying less probable actions. Nevertheless plastic methods are not yet sufficiently developed so far, for general application in dynamical or repeated phenomena.

It is considered that for seismic actions what matters above all, from an economic point of view, is to prevent a total collapse. Consequently the statements above show that limit design methods would be the most adequate. On the other hand, seismic actions have an essentially dynamic character which is difficult to take into account in a limit design method.

After all, the most adequate methods to assess the strength of a structure for seismic actions are possibly those based on its capacity to absorb energy (7). Nevertheless the investigation of this criterion is still in an early stage. While it is obviously advisable that further studies are undertaken in this field, there is no doubt that, from a practical point of view, design has yet to be based on elastic or plastic theories.

Such theories being adopted it is convenient to follow the traditional method of transforming seismic actions into equivalent static actions for which the structure is then designed by usual methods; the problem is thus decomposed and simplified.

There is no doubt that many useful informations have been obtained assuming an elastic behaviour for definition of static forces equivalent to seismic actions. It seems, nevertheless, that a design by means of these forces should not be based on an elastic but on a limit design criterion. This of course if the forces determined by the elastic methods are duly corrected. Although not the optimum solution, this duality of criteria is sufficiently close to it. It constitutes the first step for a complete analysis of the structural behaviour by means of plastic criteria.

The simplest method to take seismic actions into account consists in defining seismic coefficients, safety being secured by a suitable choice of the safety factors.

4. CHOICE OF SEISMIC COEFFICIENTS AND OF SAFETY FACTORS

In order to make sure that the safety purposes in view will not fail to be achieved, the choice of seismic and safety coefficients has to be carefully considered and, as far as possible, turned into a quantitative determination.

Let us see, in the first place, the possible policy to be followed in this choice. Two opposite criteria are possible. One consists in the adoption of small seismic coefficients and of safety factors approximately equal to the usual ones. In the other, seismic coefficients are high but safety factors low, about the unity.

In a first approximation it would seem that both criteria are to a certain extent equivalent. In fact they are not so. Adopting one or the other criterion has widely different consequences for the design. This is due to the fact that the total action to be considered is a sum of the actions due to gravity (permanent loads) with those due to the earthquakes.

Since, in seismic actions, what matters above all is that no collapse occurs, the behaviour of the structure under strong seismic forces has to be analysed, as the structure behaves quite differently in this case and under the action of small horizontal forces.

As an instance of the adoption of one or the other criterion, let us consider a reinforced concrete frame (fig. 1) made up of a beam and two columns each with a normal force N = 100 t. The initial moment at the top section of the columns can be made to vary within wide limits, according to the construction method adopted. Thus assuming that the

beam is first built as hinged and then tied together to the columns, the moments at the top of the columns are null, M = 0. If both elements are initially tied together, M may be high.

For each initial value of M = i N there corresponds a percentage of reinforcement which for usual reinforced concrete assumes the values shown in curve 1 of fig. 2. These percentages of reinforcement were computed assuming a permissible compressive stress for concrete of 75 kgcm-2.

Let us determine the percentages of reinforcement in function of the initial moments, when seismic actions are considered. The two referred criteria are applied, the action of an horizontal force at the beam, F = 2 c N being considered. The moments due to F have to be added to M in order to determine the final percentages of reinforcement.

First, a seismic coefficient c = 0.06 and an increase of the permissible stress in concrete of 1/3 are adopted. The percentages of reinforcement indicated by curve 2 are thus obtained. It is seen that for values of i < 6 cm, that is for small initial moments, no increase of reinforcement is obtained and one is limited by the minimum percentage of reinforcement indicated in the codes without considering seismic forces. As the initial eccentricity increases the corresponding percentage of reinforcement increases very fast and for i = 10 cm it exceeds by about 60% the percentage required to resist the initial moment. For the value of i = 16 cm the initial and final percentages become equal.

If a seismic coefficient c = 0.20 and a permissible stress double the usual one are adopted, curve 3 is obtained. In this case even for i = 0, a percentage of reinforcement of 1.7 % is obtained and the increase of this percentage in function of the initial eccentricity is much slower.

If instead of an elastic design we adopt a plastic design with the same value of c = 0.20, a horizontal line is obtained represented by 4 in the same fig. 2.

As the ultimate bending moments of the columns are approximately proportional to the percentage of reinforcement, this example shows that when small seismic coefficients and small increases of the permissible stresses are adopted a design is obtained that in some cases corresponds to insufficient strength to horizontal forces.

On the other hand, high seismic coefficients and high increases in the permissible stresses (leading to stresses near the ultimate point) yields a much more balanced design. If a limit design criterion is adopted, the initial moments can be neglected as in fact, due to the plasticity of the structures, they can have no influence on the value of the force F making the structure to collapse.

The preceding case is a typical instance of the following general statement. The most adequate design method, among the general criteria of adopting seismic coefficients, consists in using a limit design method with a safety factor near 1, and seismic coefficients high enough to ensure a sufficiently small probability of collapse.

Even if limit design is not adopted, it is necessary to admit high increases in the permissible stresses, the seismic coefficients being chosen in accordance with these increases.

This general design criterion adopted in some codes was also prescribed in the recently issued Portuguese code.

5. EXPERIMENTAL STUDIES

Some structural studies, static and dynamic, have been carried out in the later years at the Laboratório Nacional de Engenharia Civil with a view to determine the capacity of structures to withstand seismic actions.

Fig. 3 shows the structural model of a building being tested to investigate its resistance to horizontal forces. This is a reinforced concrete model, to a scale 1/2, of a special type of structure without cross beams. It was investigated up to what extent the moments transmitted by the columns were taken up by the slab and how efficiently longitudinal beams subjected to torsion behaved. Different model tests were carried out with slabs of reinforced concrete and of prefabricated, prestressed elements. In the latter case it was also sought to study the behaviour of connections between the prefabricated and the in situ concreted elements.

The tests showed that this structural type, by means of which ceiling beams are not apparent, could present sufficient resistance to horizontal forces if the longitudinal beams are conveniently designed to withstand torsion, notably near the columns.

Dynamic tests are at present under way on models of dams and other structures. Vibrations in these tests are applied by means of an electro-dynamical shaker which can produce forces up to 1,300 kg (3,000 lb) for frequencies ranging between 10 and 1500 Hz. This shaker is operated by a 10 KVA amplifier in which sine waves or random pulses can be introduced. For low frequencies a mechanical vibrator of eccentric masses is also used. Sinusoidal vibration tests have, above all, the purpose of studying the behaviour of structures in the elastic stage and random pulses of horizontal velocity sepectrum, reproducing real earthquakes, are specially used for studying the ultimate dynamic strength. Spectral analysis of the records is made by electric and analytical methods, an electronic computer being used in the later case.

6. CONCLUSIONS

For the design of earthquake resistant structures, the adoption of limit design methods with safety factors near 1 is recommended and justified. The values of seismic coefficients have to be selected so as to obtain a conveniently small probability of collapse. Seismic coefficients should thus be selected by means of statistical criteria taking duly into account the behaviour of the structure, the characteristics of the soil and the seismicity of the area. Strategical criteria can also supply useful decision rules notably when the available information in scarce.

The observation of real structures during and after earthquakes and model studies in which vibrations similar to those actually occurring in earthquakes are used can give, in parallel with analytical studies, a very valuable contribution towards the improvement of design methods for aseismic structures.

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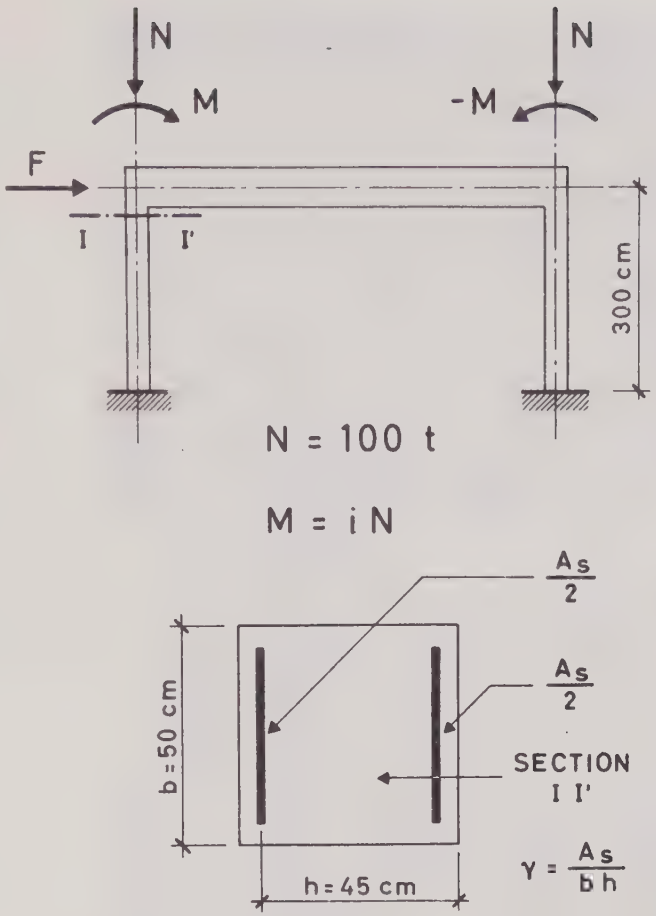
N
M
-M
N
F
I
I'
300 cm
N = 100 t
M = i N
As/2
As/2
b=50 cm
SECTION
I I'
h=45 cm
γ = As/bh

Fig. 1

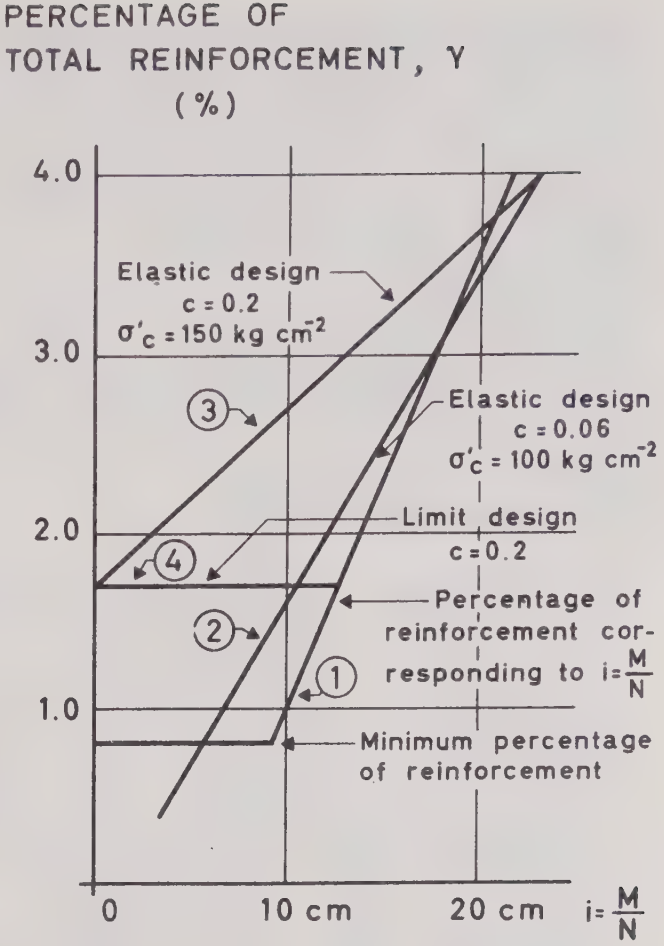
PERCENTAGE OF
TOTAL REINFORCEMENT, γ
(%)
4.0
3.0
2.0
1.0
Elastic design
c = 0.2
σ'c = 150 kg cm-2
Elastic design
c = 0.06
σ'c = 100 kg cm-2
Limit design
c = 0.2
Percentage of
reinforcement cor-
responding to i=M/N
Minimum percentage
of reinforcement
1
2
3
4
0
10 cm
20 cm
i=M/N

Fig. 2

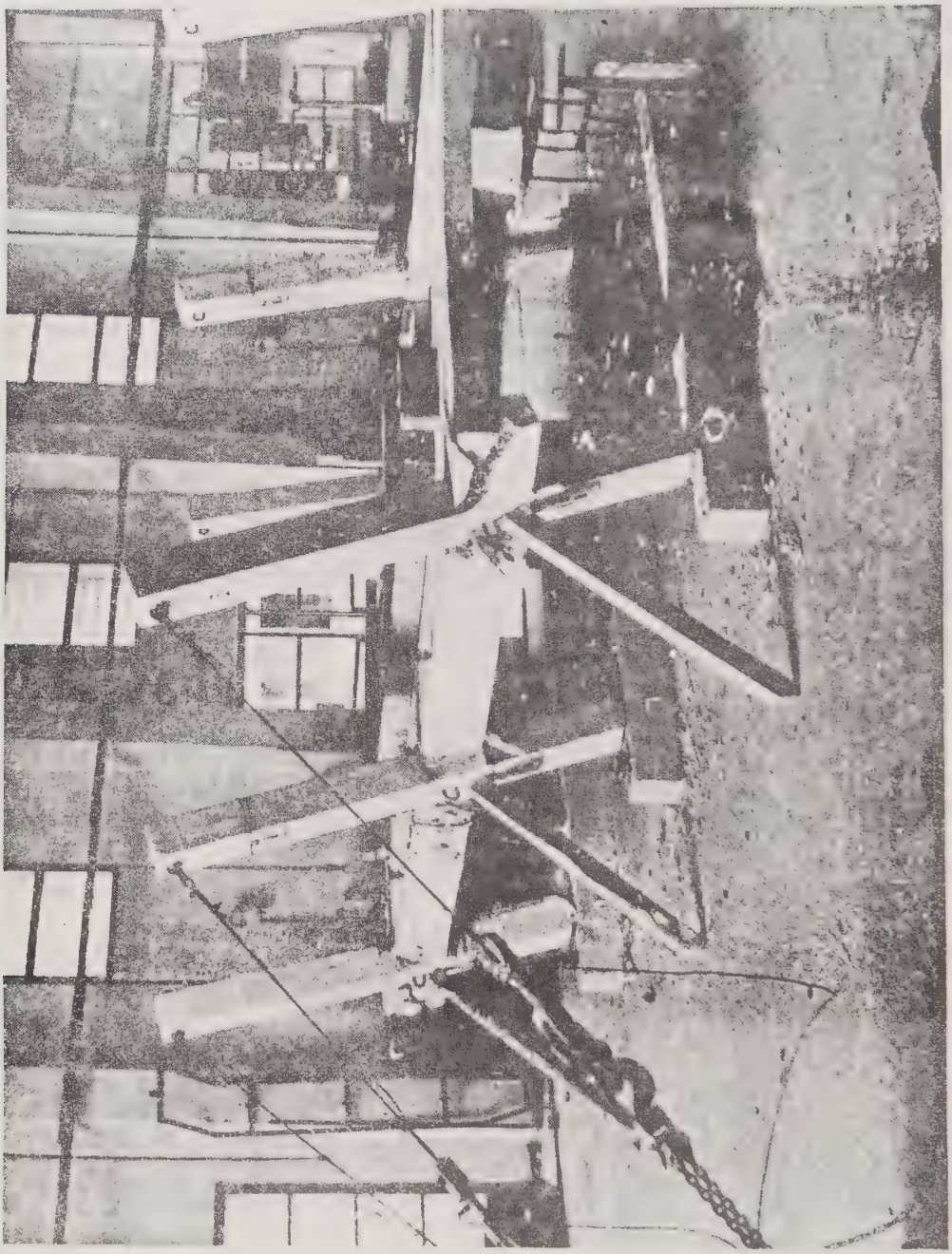

Fig. 3

DISCUSSION

G. W. Housner, California Institute of Technology, U. S. A.:

The calculated seismic coefficients for damaged buildings in Agadir are very interesting and I should like to inquire as to the method of determining the ultimate stresses used in the calculations?

J. F. Borges:

The buildings have been computed in accordance with the Portuguese code, that is, adapting for the steel and the concrete ultimate stresses corresponding to the minimum value of the yielding and compressive ultimate stress usually used. To judge the mechanical properties some samples were taken and also some measurements with a sclerometer were made. The ultimate moments were computed in accordance with the real dimensions of the elements.

It was so possible to obtain seismic coefficients for different buildings and simple structures, and these coefficients can be directly compared with those indicated in the codes.

As the slides show the behaviour of these reinforced concrete structures was very similar. At the final stage, at one storey, the curtain wall is completely destroyed and only the structure remains. Plastic hinges are formed at the top and at the bottom of the column. This shows that limit design really applies.

In most cases the seismic coefficients of the structures that did not fail range between 0.10 and 0.20, but, even so, these structures were much damaged.

THE FORECAST OF SEISMIC EFFECTS ON CONSTRUCTIONS

By

S. V. Medvedev*

INTRODUCTION

The problem of forecasting the time of earthquake occurrence has not been solved up to now. However, to project buildings and constructions in seismic regions the forecast of earthquakes can be determined on a small scale. It is necessary then to forecast what maximum oscillations can influence upon the planned building during its exploitation. The forecast of seismic effects on constructions involves the solution of three problems: 1) the seismic scale, 2) seismic zoning and 3) seismic microzoning.

1. Seismic Scale

The main aim in creating the seismic scale is to make a quantitative determination of intensity of seismic oscillations and their effects on constructions. Oscillations of ground during earthquakes are far from being harmonical. Therefore, it is necessary, first, to find methods of a quantitative estimate of non-stationary seismic oscillations. Second, to determine the very quantities of oscillations in conformity with the methods found.

It was suggested that intensity of seismic oscillations should be described with a spectrum of earthquake effects on constructions (1,2). The effect spectrum $\mathcal{Z}$ is given in the form of a product of three functions.

$$\mathcal{Z} = \mathcal{X}_0 \psi \varepsilon$$

Here value $\mathcal{X}_0$ characterizes the earthquake intensity, which stands for the displacement of the centre of a standard pendulum with fixed values of the period of own oscillations $T_0 = 0.25$ sec. and of the logarithmic decrement of the oscillation damping $\lambda = 0.50$.

Coefficient ψ determines the dependence of the effect spectrum on the period of oscillations of pendulum T, i.e. $\psi = \psi(T)$ and therefore the value ψ is called a spectral coefficient. If $T = T_0 = 0.25$ sec., $\psi = 1.0$. Coefficient $\varepsilon = \varepsilon(\lambda)$ determines the dependence of the effect spectrum $\mathcal{Z}$ on the logarithmic decrement of the pendulum damping λ. Therefore the value ε is called a damping coefficient. If $\lambda = \lambda_0 = 0.50$, $\varepsilon = 1.0$.

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Different spectral forms of seismic oscillations were suggested by investigators in different countries, no review of such papers being given here. A peculiarity of the suggested method of determining the effect spectrum is that it permitted to obtain quantitative values by an elementary statistic treatment.

To choose the values of T_0 and λ which are parameters of the spectrum, a condition was adopted that earthquakes intensity is characterized by its destructive effect. The destructive effect is determined by a damage degree of the most typical buildings. Therefore there were adopted values $T_0 = 0.25$ sec. and $\lambda_0 = 0.50$, i.e. those which correspond to buildings that are widely spread.

Seismic oscillations of a structure can be approximately described as pendulum oscillations with different periods T and logarythmic decrements of damping λ during the motion of the basement $u = u(t)$.

$$\frac{d^2x}{dt^2} + \frac{2\lambda}{T}\frac{dx}{dt} + \frac{4\pi}{T^2}x = \frac{d^2u}{dt^2}$$

The numerical solution of this equation enables us to obtain maximum values of x for different values of T and λ, function $u(t)$ being expressed graphically in the form of a seismogram and its second derivative $\ddot{u}(t)$ in the form of an accelerogram.

At $T = 0.25$ sec. and $\lambda = 0.50$, $x = x_0$ as $\psi = 1$ and $\varepsilon = 1$. Thus, the numerical values of x_0 during strong earthquakes have been found. The scale of these values was compared with a descriptive characteristics of the earthquake consequences. With respect to the division to degrees in our scale the descriptive characteristics itself is parallel to that which exists in other scale (Wood-Neumann, and Mercalli-Cancani-Sieberg). The descriptive indications in our scale are given in more differential manner than in the above-mentioned scales. The differentiation is made according to types of buildings, number of buildings, damage degree, etc.

The interpretation of materials has shown that the growth of the values of x_0 goes two times with increase of intensity J by one unit. The quantitative scale of the values of x_0 determining earthquake intensity is sited on table 1.

The scale of the values of x_0, determining the earthquake intensity was adopted as a state standard (3). The seismometer SBM which has been designed by the author to determine intensity 2 has been mounted at the seismic stations of the USSR.

The determination of the spectral coefficients $\psi(T)$ from earthquake records has been made within the range of periods from T = 0.1 sec. to T = 2 sec. The dependence of ψ on T is different for different period ranges. This dependence can be approximately expressed in the form:

$$\psi = \frac{T^2}{T_0^2}; \qquad \text{at } 0.1 \text{ sec.} \leq T \leq 0.5 \text{ sec.}$$

$$\psi = \frac{2T}{T_o} \; ; \qquad \text{at } 0.5 \text{ sec.} \leq T \leq 1.5 \text{ sec.}$$

It is important to note that for different earthquakes the values of ψ much vary and the above numerical data are mean data for ψ .

To determine the damping coefficient $\mathcal{E}(\lambda)$ average computations, analogous to those for ψ , have been performed. In addition experiments have been undertaken to test a specially designed device SD-1. This device has eight independent pendulums with equal periods of oscillations T = 0.25 sec. However, their decrements of damping ranged from 0.15 to 2.0. As a result of this, the simplest dependence has been derived (fig.1).

$$\mathcal{E} = \frac{1}{\sqrt{2\lambda}} \; ; \qquad \text{at } 0.25 \leq \lambda \leq 1.5$$

Fig. 2 gives values of the effect spectra for different periods of the own oscillations of system T. For the same systems the values of the velocities of oscillations V_T in cm/sec. (4) cited in fig. 3 have been computed with one degree of freedom.

The oscillation accelerations α_T of these systems (4) in dependence on the period of own oscillations T are shown in fig. 4.

To make computations of buildings, it is necessary to have, besides the above cited numerical values of the effect spectra, data on the expected values α of the proper oscillations of the ground. Especially important is the knowledge of the acceleration of the ground oscillations for buildings with large horizontal dimensions or for buildings with a deep-set foundation. For this purpose, the records of accelerations have been collected and generalized and the intensity in degrees for each of these earthquakes determined under a seismic scale (5).

An accelerogram represents an irregular oscillation with a variable amplitude and period. Therefore we shall conditionally call the oscillation period τ on the accelerogram a doubled interval of time between two adjoining zero values of the acceleration, and the acceleration amplitude α - the deviation value corresponding to that period.

The measured values of the periods and accelerations for the horizontal components are given in fig. 5. The graph shows a general regular growth of acceleration values with intensity increase from 5 to 8 degrees for different values of periods τ . This permitted to distinguish those parts of the graph which focus points relating to some degree. Within the range of periods from 0.1 sec. to 0.5 sec. the accelerations do not, on the average, change with the change of period. Within the range of periods from 0.5 sec. to 1.5 sec. the boundaries of the sections of different degrees are hyperbolas.

The obtained laws of the acceleration growth two fold with the increase of intensity J by one unit and the hyperbolic dependence of the acceleration on the period of the ground oscillation τ make it possible to record the following formula in which $\alpha_K = 0.0008 = 8$ mm/sec^2.

$$\alpha_o = \alpha_K 2^J \quad ; \quad \text{at } 0.1 \text{ sec.} \leq \tau \leq 0.5 \text{ sec.}$$

$$\alpha_o = \alpha_K 2^J \frac{0.5 \text{ sec.}}{\tau} \quad ; \quad \text{at } 0.5 \text{ sec.} \leq \tau \leq 1.5 \text{ sec.}$$

The statistical treatment of the data shows that the mean quadratic deviation $Q(\lg \alpha) = 0.149$ and the probability of conformity of the acceleration value with a certain degree $P(\alpha) = 0.684$.

2. Seismic Zoning

The map of seismic zoning of the USSR was compiled on the scale 1-5.000.000 (6). It divides the territory into zones according to different shock forces on the Earth's surface estimated in degrees on a seismic scale compiled in the Institute of the Earth's Physics of the USSR Academy of Sciences (1). The intensity of the estimated earthquakes is referred to the mean ground conditions.

Earthquakes are one of the manifestations of the tectonical activity of the Earth's crust. The primary role in the tectonical processes is played by vertical oscillatory motions. The vertical oscillatory motions of the Earth's crust and the tectonical activity in general are caused by deep processes occurring in the Earth's Interior at a depth of a few hundred kilometers called processes of the primary tectogenesis. The essence of these processes, the conditions of their occurrence and change in striking are far from being explained. However, the assumption is that the processes of the primary tectogenesis are caused by the transformation of large quantities of energy. At the Earth's surface these processes manifest themselves mainly in the form of mechanical motions: elevations and subsidences of the Earth's surface, folding, earthquakes. One can assume that the processes of the primary tectogenesis are followed by displacements of considerable masses of substances in the Earth's mantle. An essential part in the processes is played by a change of the volume, i.e. of density of substance that caused elevation of subsidence of the Earth's surface.

For the seismic zoning it is important that the processes of the primary tectogenesis causing vertical forces should appear on the Earth's surface on such a territory, the linear dimensions of which are commensurable with depth at which these processes take place. However, heterogeneities of plastic properties of substances of the Earth's crust in depth and striking lead to the fact that a relatively wide territory subjected to vertical displacements is different in intensity of these displacements thereby complicating the elucidation of seats of probable occurrence of earthquakes.

The methods of compiling the map of seismic zoning of the USSR of 1957 were outlined as follows. The contouring of zones with equal seismic danger was carried out on the basis of seismic and geological data, using the data on the distribution of earthquake foci in space and time, on latest tectonic motions and their relations with seismicity and many other characteristics. The intensity of the expected earthquakes within the boundaries of the contour zones was estimated from data of engineering seismology on the strongest earthquakes which occurred in the past in these

zones and their evaluation in degrees on a seismic scale.

The following data have been used to compile the map of seismic zoning: 1)the materials of the Seismicity Atlas of the USSR, as well as data obtained from the seismic stations of the USSR on the past earthquakes, 2) the data of seismogeological investigations in seismic regions of the USSR; 3) reports and materials on the inspection of destructive consequencies of strong earthquakes and results of investigations in engineering seismology.

The Seismicity Atlas permitted to more completely and accurately use the instrumental data on earthquakes which is important for all regions of the USSR and, especially, for scarcely-populated regions, the non-instrumental seismic statistics of which is poor. The knowledge of intensity, i.e. of relative energy of earthquakes, as well as of earthquake foci makes it possible to approximately estimate degrees of earthquakes in epicentres. The revealing of a peciliarity of the distribution of different-force foci enables us to use for seismic zoning not only data on destructive earthquakes but also observation data of weak earthquakes. The necessary for this data on earthquakes of last years have been taken from bulletins of the seismic stations of the USSR. The reports on the inspection of the consequencies of strong earthquakes in the past were also used.

A great volume of investigations has been undertaken in the field of seismotectonics both, to study the regional peculiarities of separate regions, and to discover general laws in the relation of seismicity and tectonics. The results of these investigations were used to insert amendments and additions to maps of seismic zoning of the territory of the USSR given in fig. 6.

On the territory of the USSR one can distinguish a few categories of regions with high seismic activity:

a) regions adjoining the boundaries of continental and oceanic zones; for example, a belt in the area of Kamchatka and Kuril Islands;

b) regions of the Alpine geosynclines on the territory of the USSR including the Carpathians, Crimea, Caucasis, Pamirs;

c) platform regions in which beginning from the Tertiary era occur intensive motions, for instance, the regions of Tien Shan and of the Baikal Lake.

On a general map of seismic zoning degrees of earthquakes correspond to mean ground conditions. The majority of populated areas and settlements are located precisely on medium grounds due to which the "zeropoint" of the map of general seismic zoning with respect to grounds stands for the most wide-spread grounds. As a rule, these are sandy soils and loams and analogous depositions.

Contributions to the work on the compilation of the map of seismic zoning of the USSR territory of 1957 on the scale 1-5.000.000 were made, directly or by their materials or papers, by the workers of many research institutes.

3. Seismic Microzoning

The destructive consequencies of strong earthquakes differ, in dependence

on the type of soils representing the foundation of the building, within the boundaries of one town. Therefore, the division of the town territory into regions of different seismicity is called seismic microzoning. The general seismic zoning of the territory of the USSR yields the figure of an expected degree of earthquake for the town as a whole. Different geological and hydrogeological conditions of the town regions may be the reason for assigning to each of them greater of lesser intensity by one degree.

It was established that the change of intensity degree may take place due to: a) difference of elastic properties of grounds, b)presence of ground waters, and c) difference in the depths of soil layers. The methods of seismic microzoning were worked out on the basis of the materials obtained from: 1) the study of the consequencies of strong earthquakes, 2) the measurement of oscillations at different grounds during weak earthquakes, 3) the study of ground oscillations caused by artificial sources.

It was established (7) that the increase of earthquake intensity n_i during the transition from some ground conditions to some others can be determined from the following empirical formula:

$$n_i = 1.67\left[\lg(v_o\rho_o) - \lg(v_n\rho_n)\right] + b\cdot e^{-0.04h^2}$$

where: n_i is increment of the earthquake intensity expressed in degrees of a seismic scale for soils with v_n and ρ_n with respect to soils having v_o and ρ_o ;

v_n, v_o - propagation velocities of P elastic waves in soils of natural moisture;

ρ_n, ρ_o - soil densities;

h - depth of the ground waters level in m.;

b - coefficient depending on the type of ground;

for gravel $b = 0.5$; for clay and sandy soils $b = 1.0$.

Basing on experimental data in conformity with the above formula computations have been made of the earthquake intensity n_i for different soils. The nomenclature of points has been adopted in a form adopted in construction practice, say, in determining permissible stress on soil.

The values of increments of earthquake intensity n_i cited in the table have been determined with respect to granite. It is adopted that for granite $n_i = 0$, so for all other soils n_i have positive values. While carrying out the seismic microzoning the characteristics of medium ground conditions (n_i) are being determined. They correspond to a figure of seismic intensity established by the general seismic zoning of the USSR. The difference between the increments (n_i) n_i serves as a basis for compiling the map of seismic microzoning, tenth parts of the intensity degrees being rounded up to integers of degrees.

The values cited in fig. 3 can be made more accurate from data of temporary and portable seismic stations. In dependence on the type of soils there was introduced correction factor of intensity K_i used to take into account seismic effects on structures. This intensity factor K_i the values of which are given in tab.3 is multiplied by the horizontal seismic force of inertia. If intensity is determined from the map of seismic zoning, the effect spectrum β and acceleration α_o must be multipled by factor

K_i depending on the soils.

CONCLUSION

The described quantitative solution of the problem of forecasting seismic effects on constructions including the seismic scale, seismic zoning and seismic microzoning is now only approximate and requires additional accuracy and progress. To improve the forecast of the seismic effects, it is necessary, firstly, to extend the investigations in the field of engineering seismology, and, secondly, to coordinate the efforts of research institutions of different countries.

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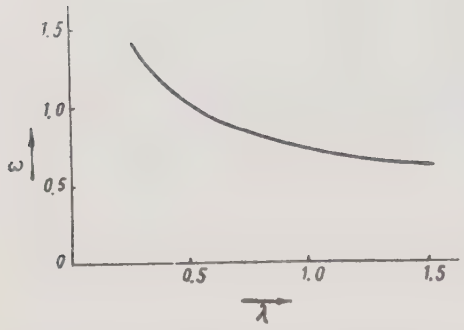

Fig. 1 The graph of the damping coefficient $\xi = \xi(\lambda)$

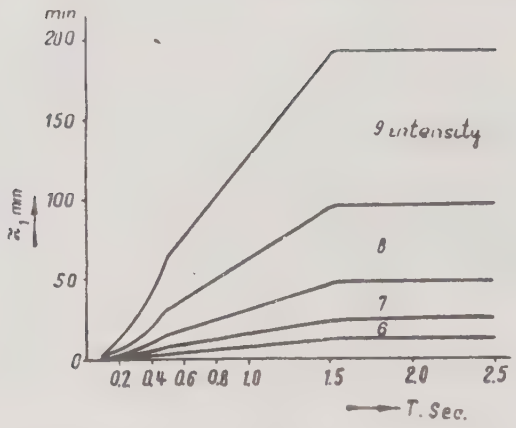

Fig. 2 The value of the effect spectra $\mathcal{K}$ for earthquakes of 6 to 9 degrees at $\xi = 1$.

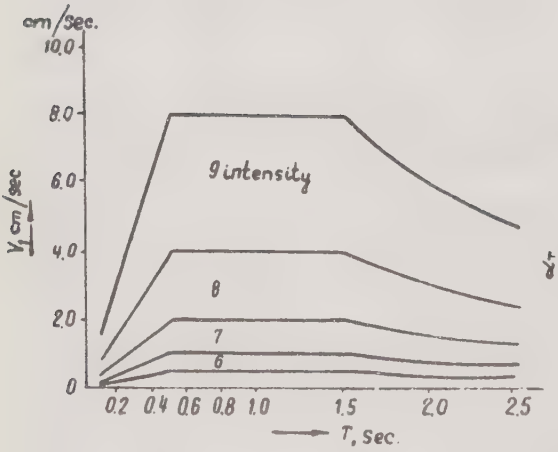

Fig. 3

The values of the oscillation velocities V_T in cm/sec at $\lambda = 0.5$ for a system with one degree of freedom.

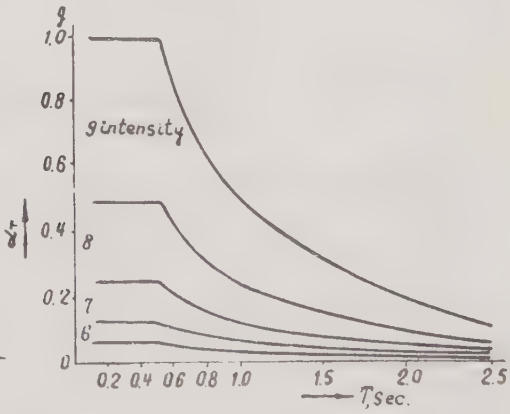

Fig. 4

The values of the oscillation accelerations α_T in parts from g for a system with one degree of freedom.

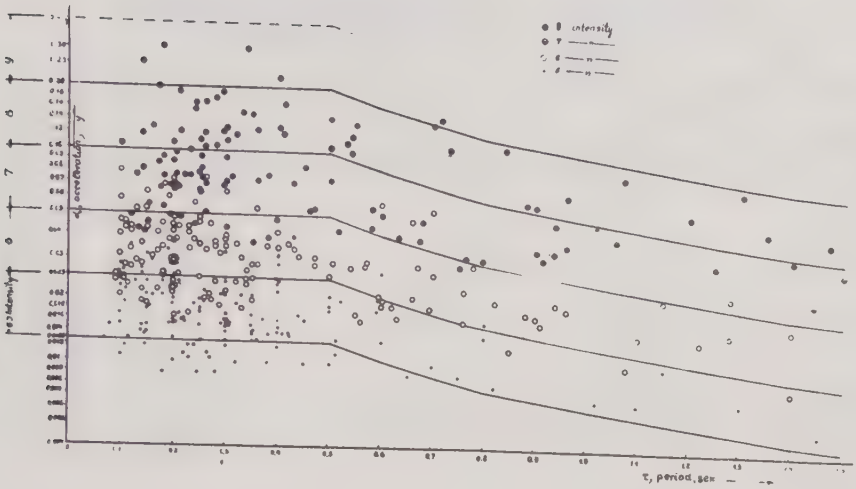

Fig. 5 The ratio of accelerations and oscillation periods of ground during earthquakes of different degrees.

Fig. 6 The map of seismic zoning of the USSR of 1957.

Table 1

The value of x_0 in mm of the displacements of a spherical elastic pendulum at $T_0 = 0.25$ sec. and $\lambda = 0.50$ for different intensities.

J, degree	x_0, mm	J, degree	x_0, mm
4	< 5	8	4.1-8.0
5	0.5- 1.0	9	8.1-16.0
6	1.1- 2.0	10	16.1-32.0
7	2.1- 4.0	11	> 32.0

Table 2

The accelerations of the ground oscillations α_0 at periods from 0.1 sec. to 0.5 sec.

J in degrees	Accelerations in parts of gravity g
6	$0.025\,g < \alpha_0 \leq 0.05\,g$
7	$0.05 < \alpha_0 \leq 0.1$
8	$0.1 < \alpha_0 \leq 0.2$
9	$0.2 < \alpha_0 \leq 0.4$

Table 3

The values of increments of earthquake intensity n_i and intensity factors K_i

Name of soil	v, km/sec	n_i, degrees	K_i
I. Rocky Soils			
Granites	5.6	0	0.3-0.4
Limestones, shales, close gneisses	3.5-4.5	0.2-0.4	0.4-0.5
Close sandstones	2.2-3.0	0.5-0.8	0.5-0.7
Limestones, shales, disturbed sandstones	1.5-2.3	0.7-1.1	0.6-0.8
II. Semi-Rocky Soils			
Gypsum	2.4-3.0	0.6-0.8	0.6-0.7
Marls	2.0-2.6	0.7-1.0	0.6-0.8
Cemented sands	1.4-1.9	1.0-1.2	0.8-0.9
III. Large-Fragment Soils			
Crushed-stone soils and coarse gravels	1.3-2.1	0.9-1.3	0.7-0.9
Gravels(from crystalline rocks)	1.2-1.9	1.0-1.4	0.8-1.0
Gravels(from sedimentary rocks)	1.1-1.7	1.1-1.5	0.8-1.1
IV. Sands			
Gravel and coarse-grained soils	1.1-1.6	1.2-1.4	0.8-1.0
Medium-grained sand	1.0-1.4	1.3-1.6	0.9-1.2
Fine-grained and pulverized sands	0.7-1.2	1.4-1.8	1.0-1.3
V. Clay Soils			
Clays	0.9-1.5	1.2-1.6	0.9-1.2
Loams	0.8-1.4	1.3-1.7	0.9-1.2
Sandy Loams	0.4-1.2	1.4-1.8	1.0-1.3
Sandy Loams and weak Loams	0.5-0.8	1.7-2.1	1.2-1.6
VI. Made and Subsoil Grounds			
Made Soils	0.3-0.5	2.3-2.6	1.9-2.3
Subsoils	0.2-0.3	2.6-3.0	2.3-3.0

Name of soil	v, km/sec	n_i, degrees	K_i
VII. Watered Soils			
Gravels and Coarse Gravels	-	1.6-2.0	1.2-1.5
Sands	-	2.0-2.4	1.5-2.0
Clays (Loams and Sandy Loams)	-	2.4-2.8	2.0-2.6
Made Soils and Subsoils	-	3.3-3.9	3.7-5.6

EARTHQUAKE RESISTANT DESIGN OF SMALL BUILDINGS

By

HENRY J. DEGENKOLB[1]

The problems concerning the design of small buildings to resist lateral forces have been largely neglected in the engineering literature. Most of the research and the development of mathematical tools for studying earthquake response have concerned themselves with the larger structures or the structures whose dynamic properties can be rather simply expressed in mathematical terms. In addition to this, engineers and architects in most parts of the United States, at least, even today do not consider lateral loading on small structures to be an engineering problem, and many building codes completely ignore it. From the newspaper accounts of building failures in various parts of the world, it is probable that this same situation with regard to lateral loads on small buildings is common in other countries. It is somewhat strange that this condition should exist, since in every country in the world, without exception, the small buildings make up the vast majority of the total construction. This fact alone should concentrate research and study on the problems of the small building for two important reasons. First, the majority of the population live and work in and are otherwise concerned with the safety of small buildings. More lives are lost in earthquake failures of small buildings than in any other kind. Secondly, more total construction cost is involved in the construction of the small buildings than in any other class of construction. As a consequence, research into possible economies in the design or construction of small buildings offer more possibilities of fruitful return than in any other construction field.

Let us consider some of the fundamentals upon which any engineering design is based. In approaching the design of any specific structure there are two general basic approaches; the first of these is the trial and error approach - the proof of time. In using this approach if it is necessary to span a certain distance with a wood joist floor, a certain size joist is tried. If it falls down or deflects too much, a larger size is used and the process repeated until a satisfactory floor is obtained.

Long before we had our formulae for bending moments, maximum stress, etc., builders were constructing bridges, buildings, towers, domes and other structures. If one fell down, that was unfortunate and the next one was built differently, or heavier, and eventually one was built that stood up. Experience was gained and empirical rules of thumb were developed and major structures resulted. Some of the longest timber bridges ever built were built under this method. The domes of the old cathedrals were built in this manner and many are still standing today after centuries of use. It is not a method to be dismissed lightly, as it formed the foundation for our engineering profession.

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The second approach to structural engineering problems consists of determining the forces that exist, and by applying certain laws of stress and the ability of a material to resist that stress, determine mathematically what structure is needed to resist that force with an adequate factor of safety. Instead of just applying a load to a floor joist in order to see if it will stand up, we now calculate a bending moment, and by using an appropriate fibre stress in a formula based on Hooke's Law, we determine the proper size of joist that will safely carry the required load. When all factors, relationships and data are known, this second method, of necessity, must agree with the first elementary method. When we do know all the factors, we can more closely predict the results that can be obtained only expensively and laboriously by tests.

We engineers are mathematically inclined by nature or we would not have survived the type of training we were given, and we therefore appreciate and endeavor to use this second method. While this second method is certainly valid and appropriate when all factors are known, it can certainly be misleading if certain portions of the data are not known. Yet, from our basic inclination, at times we endeavor to apply the second method before all the data is available.

Now just how do these basic approaches affect our lateral force design? At the present time our lateral force design is based on the first method. Coefficients are based on observation and past experience. The requirements as to parapets, fastening of walls and ornaments, foundation ties and general attitude of design are all based upon observation of structures that either survived a destructive earthquake or were demolished by it.

In recent years a large amount of effort has been expended in gathering data for applying the second method to the art, not science, of seismic resistant design. Several agencies in various countries have developed, installed and maintained instruments to measure basic earthquake motion and several good records are now available. Various investigators have studied the application of this data to idealized simple structures of various types. Studies have been made of how the findings on these idealized structures may compare with actual prototype structures. The records of earthquake motion indicate ground accelerations of the order of magnitude of 30 to 40 percent of gravity, or even more. By applying the spectrum response of idealized structures to various earthquake records, Professor Housner[1] has found that this has been amplified through resonance to accelerations of the order of several hundred percent. Studies of internal damping and some foundation effects have reduced this theoretical maximum again to the order of almost 100 percent of gravity. However, studies to date do not lower these theoretical figures appreciably and there is increasing pressure, in the United States, that our building code design values should be increased to a level more closely approaching this theoretical figure. In the face of these measurements and studies, earthquake experience tells us that many well designed structures have withstood major earthquakes with rather nominal percentages of calculable earthquake resistance. We have seen numerous instances of steel or timber buildings having negligible resistance to lateral forces coming through major earthquakes with absolutely no damage at all.

Certainly, to date, the results obtained by the two approaches as related to earthquake design do not agree. Based only upon observations, a rather small coefficient is adequate for many types of structures, and our theoretical second approach has not yet developed to the point where these small coefficients can be explained in useable mathematical terms.

Before proceeding further, it should be made clear that I am not disparaging any research that has gone before or is to be done in the future. If we are ever to graduate from the first crude approach to design to a true engineering approach, we will need all research possible in all directions that indicate any promise. This does not change the facts, however, as regards present day knowledge and conditions.

A fundamental characteristic of small buildings as a class is the diversity and complexity of their construction, both as regards to material, shapes, and methods of framing. Large structures are usually constructed with structural steel or reinforced concrete as the basic structural material. Small structures, on the other hand, may be built of almost anything. In the United States, timber is probably the most common construction material. It is often used as the basic structural material but it is also often used in conjunction with other materials such as masonry units in its various forms, and all of the other basic materials. While timber is basically an elastic material, when fastened together in assemblies commonly used for lateral resistance it becomes highly plastic, especially when the commonest connectors, nails, are used. As Jacobsen found in conducting dynamic tests upon timber shear walls[2] "It was soon found that the action of the nails was such that the individual tests could not be repeated on the same model with the same results; in other words, the elastic characteristics of the models, due to the loosening of the nails, changed considerably from blow to blow."

Lateral load tests on diagonally sheathed shear walls and roof diaphragms also indicate that a considerable variation in deflection may exist in repeated static loadings. Many of the other building materials common to small structures exhibit the same tendencies to a greater or less degree. The difficulties of determining definite dynamic properties of a single building composed of several different materials, none or few of them elastic, are almost insurmountable.

Small buildings come in an infinite variety of shapes and framing schemes. There is no such thing as a typical small building, but Figures 1 to 3 give some indication of the range of complexity that is characteristic of this entire class of structure.

Figure 1 shows a small bank building using a wood roof with concrete exterior walls. The lower level roof and the mezzanine diaphragms have very large span depth ratios but by creating shear walls connecting them with the roof diaphragm and tying together with moment connections, the deflections could be held within acceptable levels. It would be very difficult to determine the dynamic properties. Figure 2 shows a building containing a bowling alley with the floor at five different levels, a small mezzanine and various relative roof elevations. Any dynamic properties determined at one end of the building would have little relation to those found at any other location. Figure 3 shows an all concrete building housing a public utility. The

complex shape was required by the shape of the lot as well as the peculiar requirements of the equipment and economic factors. The thought of determining the various periods of this structure for a theoretical dynamic analysis is appalling.

It can be seen from the above that, since our research and theoretical analysis of earthquake response has not developed to the point where it is practical for use in the small buildings, present practice relies almost entirely on the first, trial and error, method of analysis. Our methods of providing lateral force resistance in small buildings is empirical, based upon observation of many buildings that have performed adequately in past earthquakes as well as those that have failed. It is common to provide for 50% or 100% G for parapets for example since observation tells us that they are extremely vulnerable to earthquakes. Moreover, any failure of a parapet creates an especial hazard on sidewalks and public ways and so extraordinary precautions are necessary to protect the public.

The same precaution, to a somewhat lesser degree, is observed in the attachment of walls since past earthquakes have indicated that many walls have fallen out of buildings, endangering the public.

The coefficients chosen for the design of the building are averages of the judgments of the engineers who have observed the effects of past earthquakes. Undoubtedly, in specific cases such as the flexible buildings illustrated in Figures 4 and 5, these may be too high, while in other cases there is quite a possibility they are too low. But the range of the problem with small structures is so great and the indeterminate factors so overwhelming, that it is the best that can be done practically at the present time. One non-scientific factor is also of major importance when evaluating our design methods. Almost without exception, small buildings are restricted economically. There are few monuments built today and the purpose of creating the building is to return some benefit to the builder, often monetary, through rent or other commercial usage. This restriction, through economy, extends to the design phases of the construction. The engineer or architect must employ some method in design that will turn out an economical safe structure with a minimum of analysis consistent with the problem at hand.

Now, in view of the above general background, we must evaluate some of the requirements necessary for adequate design to resist earthquakes. First and foremost, the whole attitude of the engineer must be changed somewhat. In vertical load design, the loads are known. The designer can analyze his loads and how they are to be carried with considerable certainty. From years of research, development and study as a background, the requirements of the actual building can be readily estimated and the performance of the building and its resultant factor of safety can be predicted with a reasonable degree of precision. To a somewhat lesser extent, the same situation exists in designing for wind loads. When considering earthquake effects, however, the direct opposite is true. The effective loads on the building are not known and cannot be predicted. In every instance, the designer must bear in mind that his assumed forces - the forces specified in building codes - may be considerably exceeded at least in parts of his structure. The solution to

this problem does not lie in choosing high design coefficients which would make the small structure economically impossible. It is rather in the attitude of the designer in providing a tough structural system which can resist shock well, even though it may not withstand the high static forces often assumed. One of the most typical and important results of this approach is the proven necessity for tying the various parts of the structure together. Probably the commonest immediate cause of structure failure in an earthquake is the lack of adequate coherence of the structural parts as illustrated by the disintegration of unreinforced unit masonry, the falling off of parapets, collapse of walls, and separation of wings from the body of the structure.

Second in importance only to the attitude of the designer, are the materials of construction and the related framing scheme chosen. Certain materials and framing methods have considerable inherent resistance to earthquake loadings. Figures 4 and 5 illustrate certain buildings that had no earthquake damage at all in spite of having negligible calculable lateral resistance. They went through the same earthquake that caused the failure of the building shown in Figure 6 which used materials and details proven to be particularly vulnerable in many earthquakes.

There are two basic method of providing for the horizontal forces created by the earthquake ground motion. In specific structures, the designer may choose one method or the other depending upon conditions, but, because of specific utilitarian requirements it is probable that combinations of both methods will be required in many structures. For simplicity here, we will consider only each specific method and not the combination. The first method resists these forces through moment connections or knee braces as illustrated in Figure 7. This is typical of many types of mill structures, light sheds and other places where shear walls cannot be used for architectural or utilitarian reasons. The forces are carried to the ground through bending of the columns and girders. This bending resistance must be developed at the joints.

This method has the advantage that large open spaces can be created without interference from permanent bracing walls. It is often forced upon the engineer in order to achieve the open effect of large glass areas prevalent in modern architecture. The framing can absorb large amounts of energy and can probably withstand a larger earthquake with less chance of total collapse than any other type of framing. It has two major disadvantages. In general, it is a more expensive method of framing since the member sizes are often larger than those required by vertical loads alone and the connections are more complicated and hence more expensive. However, its major disadvantage lies in the fact that it is usually somewhat flexible. This flexibility denies the proper support of the more expensive finishes common to many of our buildings. Plaster, tile, masonry, and veneers of all types are usually quite rigid and brittle and must be supported with a minimum of deflection.

The second base method of providing for lateral forces is through shear walls and diaphragms. The action in this method, illustrated in Figure 8, resembles the action of a box. In most cases, this is the most economical

method of providing for lateral forces. It is by far the stiffest method and therefore gives the best protection against property damage especially to brittle finishes such as plaster. Its economy lies in the fact that it utilizes building components such as walls, roofs and floors already provided in the building. In the United States, this method of bracing is used in the vast majority of the small buildings. It is so prevalent, and the engineering literature is so meager, that it would be well to review the basic design principles here.

Consider the idealized simple structure shown in Figure 8, where we have a simple, box-like, one-story structure. The inertia loads of the side walls (or wind pressure and suction in the case of wind) act on the walls. The wall framing spans from the foundation to the roof as a beam and must be designed as such in addition to the column action due to the vertical loads in the roof. The top of the wall thereby produces horizontal loads against the roof framing. The wall must be attached to the roof framing strongly enough to resist the forces involved. The roof framing now acts as a large girder taking the horizontal reactions of the walls and the roof's own inertia horizontal loads and spans to the end walls which must resist these reactions.

The roof system that carries these loads is called the diaphragm. In some cases, it is actually framed with diagonal rods or angles to form a bracing truss, which is the only case generally considered in engineering literature. From observation, a few tests, and considerable engineering intuition, the custom has developed of using the actual roof framing, that is, the wood or plywood sheathing, concrete or metal deck or other construction materials to serve as the structural element to resist these lateral loads. How is this done?

Considering this roof diaphragm as a girder with the external loads shown, we see that there are the usual shear and moment forces to resist. Generally, chords are provided to resist the moment stresses. The roof material itself resists the shear stresses. This sounds simple, and in some materials such as with concrete or plywood, it is.

In Figure 9, the roof is spanned in the short direction with roof rafters, while sheets of plywood form the roof sheathing. The stress action is similar to that of a large plate girder with the plywood resisting the shear forces and the continuous chords on the long sides resisting the moment forces. It is usually assumed that the web does not help to resist the moment forces. The nailing of the plywood is calculated from the shear forces similar to the web rivet design for a steel plate girder. Many tests of this type of construction have been made in the United States by the Douglas Fir Plywood Association.(3)

Figure 10 illustrates the type of diagonally sheathed diaphragm which has been tested most often to date. Here, too, the roof is spanned in the short direction by roof rafters, while sheathing boards span the space between rafters at a 45° angle. These sheathing boards are usually 1" x 6" or 1" x 8" boards nailed at each joist with two or more 8 penny nails,

depending upon the calculated stress. Again, the continuous chords on the long sides resist the moment forces, while the diagonal sheathing resists the shear forces. As shown in Figure 10, the sheathing boards are in tension, but since lateral loads may come from any direction, the loading can be reversed and the boards would be in compression. Although there may be considerable bending in the sheathing boards, tests verify (4,5) that the primary load resistance in an efficient diaphragm is due to the axial stress in the sheathing boards. The nailing is calculated from this axial stress. As may be seen from Figure 10, there is a component of stress perpendicular to the end posts that puts it in bending. The end post must be connected to the longitudinal chords adequately to resist this stress component. There are similar bending components perpendicular to the long continuous chords, but as in this example, the effect is usually minor because of the comparatively small spacing between rafters, which resist these perpendicular components.

Figure 11 illustrates a similar type of framing, using a double layer of diagonal sheathing to resist the shear forces. This system has not been tested since 1935.(6) It will be noted that in this system one layer of sheathing is in tension while the other layer, at right angles to it, is in compression. The forces perpendicular to the end posts cancel each other and so no bending tendency results. This system can be completely analyzed by simple statics and the maximum strength is limited only by the nailing room provided at the various contacts and intersections. Structures at the Golden Gate International Exposition in 1939 successfully resisted shears of over 4,000 lbs. per foot by this method.

Figure 12 shows a most common method of framing a roof which, to date, has not been tested except for one full size test in Los Angeles in 1952.(7) However, by comparison with tests on the diaphragm shown in Figure 10, the following inferences may be drawn:

The loads from the side walls are delivered to the longitudinal girts which span to the trusses. The end reactions of the girts are delivered through the trusses to the diaphragm, usually through blocking over the truss chords. The girts are spliced for axial load to provide chords to resist the moments on the diaphragm as a whole. The shears on the diaphragm are resisted by the sheathing boards, again primarily by their axial loads, similar to those illustrated in Figure 10.

In this case, the force components causing major bending stresses in boundary members occur at the girts. It can be seen in the figure that as the sheathing boards take their axial tensions as shown here, the girts are pulled inward, similar to the end posts of Figure 10, and must be designed for this bending force. The axial forces from the sheathing at the end walls also tend to pull these walls in, but here the span is only between purlins. The purlins should be spliced throughout the length of the diaphragm to balance the similar forces at the opposite end of the building.

The tests to date and the analysis so far have assumed that the diaphragm lies essentially in a plane so that forces normal to the sheathing

may be neglected in analyzing the diaphragm action. However, it is common practice to use either bowstring trusses or pitched trusses of one type or another and, consequently, the diaphragm is either pitched or curved. Some studies have been made indicating the secondary forces that may be caused by pitching or curving the diaphragm.(8) Similar tests have been run on diaphragms of other materials such as metal decks and poured gypsum.(9)

At this stage of the discussion, it is appropriate to consider what properties are desirable in a diaphragm that will make it most suitable to resist earthquake forces. Naturally, since it is serving as part of the structure, it should be adequately strong to resist the normal vertical loads, it must not deteriorate with time or weather, and it must be reasonably economical in its application. The diaphragm must have some reserve of strength so that it can act after normal deflections have taken place. For example, in Figure 13 when reasonable foundation settlement takes place or truss deflection occurs, its value should be relatively unimpaired. The assembly as a whole must be capable of some degree of analysis in order to predict its strength, and its strength must be proven by test. And the diaphragm must be able to resist deflection.

As seen in Figure 14, the loads on the diaphragm will cause it to deflect. This deflection allows the walls, in turn, to deflect. If the wall is stiff, is short, is brittle, or is fixed at the base and the diaphragm deflects too much, the wall can be cracked.

By assuming the wall to be fixed at the base, it can be shown that

$$\triangle = \frac{96H^2f}{Ed}$$

where $\triangle$ is the deflection in inches, f is the fibre stress in pounds per square inch and E the bending modulus of elasticity, H is the height of wall in feet and d is thickness in inches.

In a condition where the wall is fixed as by a loading dock in Figure 15, or by construction in a story below, this or a similar formula may be very useful in determining the limiting deflections. However, no stretch of the imagination could indicate any validity in the other cases shown such as in a wood stud wall where rotation takes place in the plates and not bending of the studs, or in a masonry wall where the foundation construction does not fix the base. As a result, larger deflections might safely be allowed than indicated by the above formula.

At present, there are no suitable criteria regarding the desirable properties in a diaphragm. As can be seen, the designed faces a dilemma in choosing the proper material for diaphragm action. On the one hand, as can be seen from the discussion above, a rigid diaphragm is desirable and may even be necessary to provide adequate support for the various walls and finishes. On the other hand, it appears that a little flexibility aids a structure to survive a major earthquake, especially when the damaging

"forces" are not known. In fact, the loads may vary inversely with some function of the flexibility. We have seen instances where well designed buildings with rigid elements have failed in such a manner as to indicate that it required a large force to cause failure. Permitting the diaphragm to deflect as in Figure 14 relieves the load on the diaphragm because of the flexibility. And we have numerous instances of where this flexibility must have actually permitted buildings with very little lateral resistance to successfully weather rather large earthquakes. So it is quite probable that some flexibility is desirable, but how much is the question. The flexibility also permits a secondary action with regard to walls and parapets, probably increasing the amount of strength and attachment necessary.

At the present time, there is no accepted criteria for judging the relative merits of diaphragms and each case must be evaluated by the structural engineer based primarily on his judgment. It is to be hoped that some reasonable criteria as to strength, rigidity, "toughness", all as related to proper design coefficients, can be determined soon by some responsible group.

Shear walls can be designed in a somewhat similar manner as the roof diaphragms. It is important that all stress transfers are adequately provided for in the design and that the overturning forces are taken care of. The overturning forces can be quite a problem, especially when walls of light construction materials are involved. Tests of timber shear walls have been conducted by the Forest Products Laboratory.(10) These tests reveal the importance of proper details at corners and joints if a strong, rigid wall is to be obtained.

It may be assumed from the foregoing discussions that the art of design of small buildings for earthquake forces is in a very elementary and unsatisfactory state at the present time. It is true that it is in an elementary state as compared to the status of vertical load design. However, our empirical methods of design have furnished a reasonable basis for many practical design problems because it has been based on observation.

Just because the design method is empirical does not, per se, mean that it is poor. Even with its faults, it satisfies a need that is present now. Many structures that have performed satisfactorily in recent earthquakes have been built by these methods. The empirical method now used has the major advantage that it is rather simply applied and can be used on the most complex structures. It is a method that is suitable in the average design office because it can be quickly and economically used, and on small structures especially, this is an important factor.

In 1933, as a result of the Long Beach earthquake(11, 12), the State of California enacted the so-called "Field Act"(15) requiring that public schools built thereafter in California be designed for earthquake forces to insure safety of construction. Since that date, there have been several destructive earthquakes in California, notably Imperial Valley, 1940, (14), Kern County 1952, (15,16), Eureka, 1954, (17), and San Francisco, 1957, (18). In these earthquakes, there was considerable damage to the older schools

built before 1933 and in other structures as well. Although many new schools had been bult in the area of major damage, not one school constructed under the Field Act suffered any structural damage to the degree that would have created a hazard to life or limb. In a few notable examples, pre-Field Act schools, badly damaged, were adjacent to or very near newer schools that were completely unharmed. In general, it can be stated that any of the small buildings built in accordance with the various building codes in California, empirical and crude though they may be in their earthquake requirements, performed very well.

CONCLUSIONS:

From the general discussion presented above, the following conclusions can be drawn:

1. Our present lateral force coefficients and design practices are generally based on observations of structural behavior. Many of the old buildings used rather flexible diaphragms. New materials should have basically similar mechanical properties to those used in the past in order to use the same coefficients. If the mechanical properties of new materials are substantially different, other and possibly larger coefficients may be necessary to assure safe construction.

2. The art of theoretical analysis has not yet developed to the point where mathematical predictions of structural behavior will agree with the actual behavior in major earthquakes.

3. When loads are known, such as wind forces, diaphragms should be as rigid as possible.

4. Under seismic and shock loadings some flexibility or at least toughness of a structure is desirable, based on observations of past performances.

5. Diaphragms are useful, economical tools in designing structures to resist lateral loads.

6. Diaphragms must serve to economically resist vertical loads, without deterioration through exposure, and must have sufficient reserve strength to function adequately in spite of secondary stresses caused by normal deflections, foundation settlement and other circumstances.

7. It is of great importance that the various components of a structure be adequately tied together so that the structure will act as a unit.

8. In the final analysis, the producing of safe earthquake-resistant structures for the public depends basically on the experience and

knowledge of the designing engineer rather than on detailed building-code requirements. The engineer who knows the requirements of a project must choose the proper materials for each specific job. He must connect the materials together so as to provide a coherent unit with sufficient lateral resistance to satisfy the particular requirements and yet he must maintain strict economy for his client. The problem is too complex for the provisions of a "handbook" building code.

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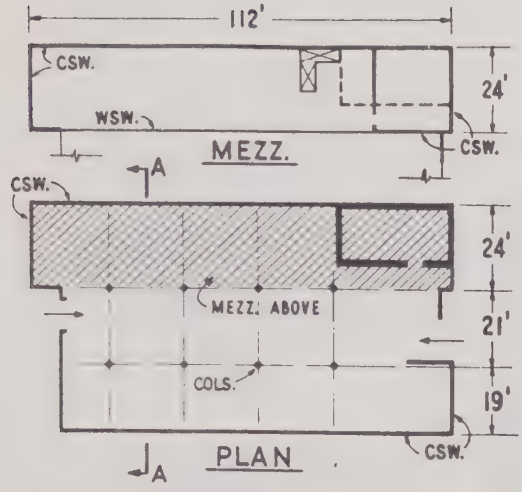

CSW. = CONC. SHEAR WALL
WSW. = WOOD SHEAR WALL

FIG. No. 1

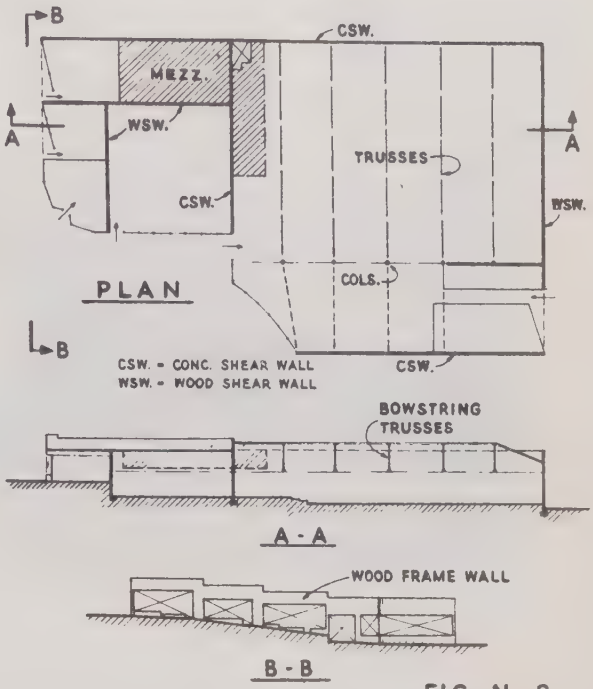

FIG. No. 2

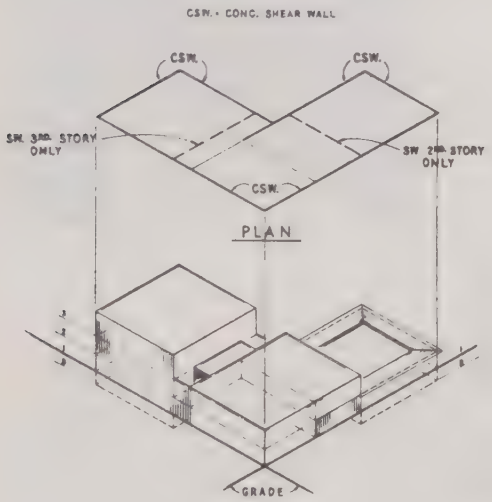

FIG. No. 3

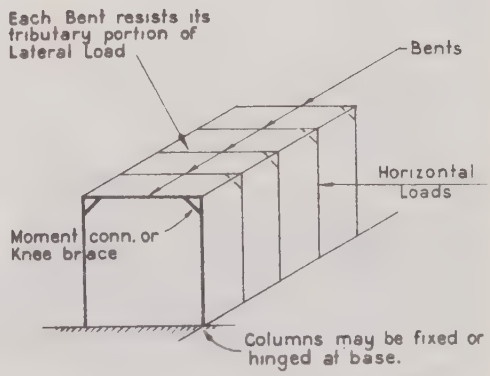
Each Bent resists its tributary portion of Lateral Load
Bents
Horizontal Loads
Moment conn. or Knee brace
Columns may be fixed or hinged at base.

FIG. No.7

FIG. No. 4

Corrugated Iron Warehouse, Tehachapi, July 21, 1952. This structure has a timber frame without bracing and was not damaged in the 1952 Earthquake.

FIG. No. 5

Clark Hotel, Tehachapi, July 21, 1952. This old Board and Batten wood structure several inches out of plumb and with no calculable bracing was undamaged by the 1952 Earthquake.

FIG. No. 6

Lodge Hall in Tehachapi, July 21, 1952. Ordinary construction using unreinforced brick walls and wood roof. The roof over the 2nd. floor Auditorium now rests upon the piano and chairs after the walls collapsed.

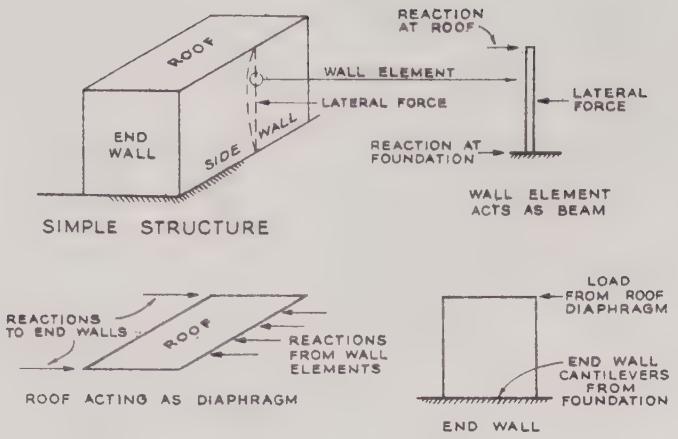

FIG. No. 8

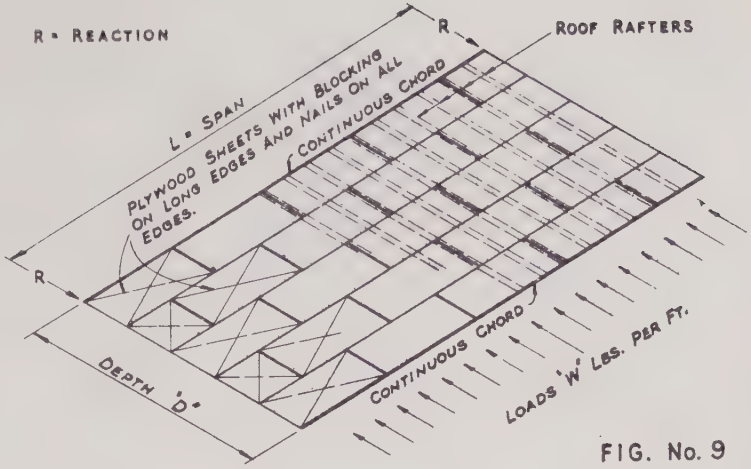

FIG. No. 9

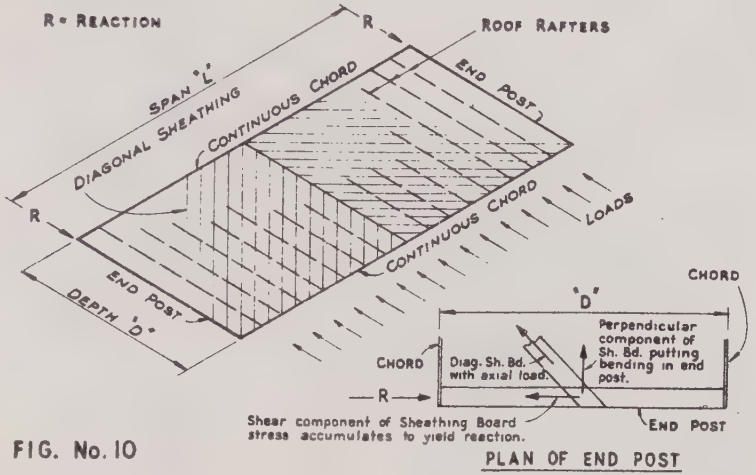

FIG. No. 10

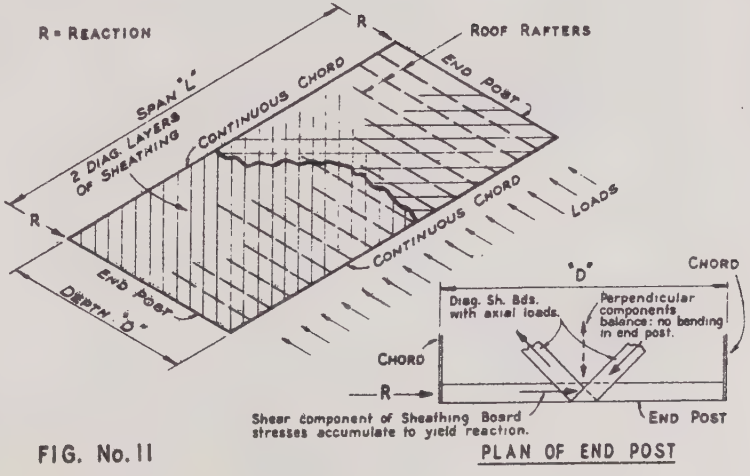

FIG. No. 11

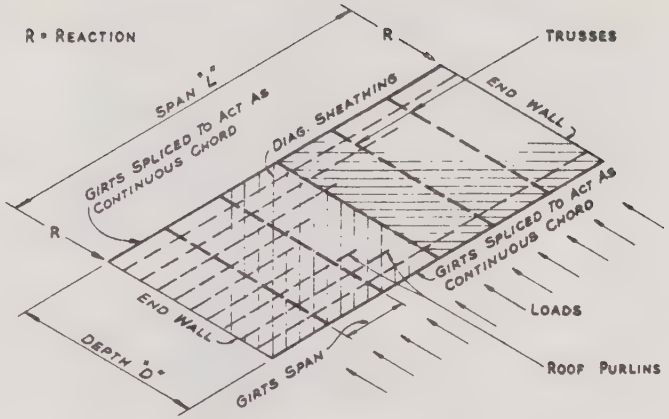

FIG. No. 12

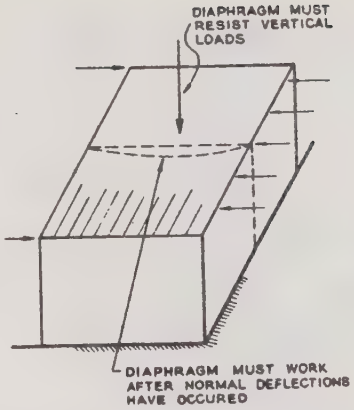

FIG. No. 13 A

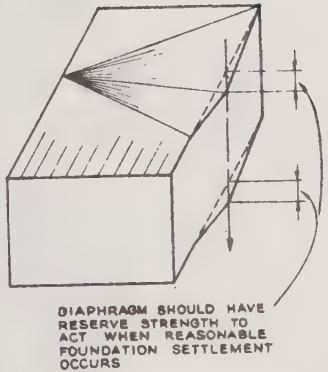

FIG. No. 13 B

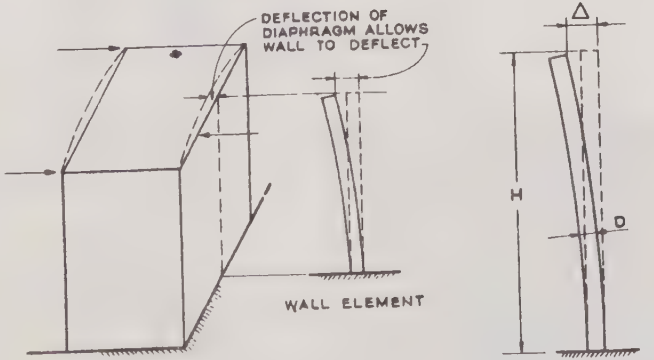

FIG. No. 14

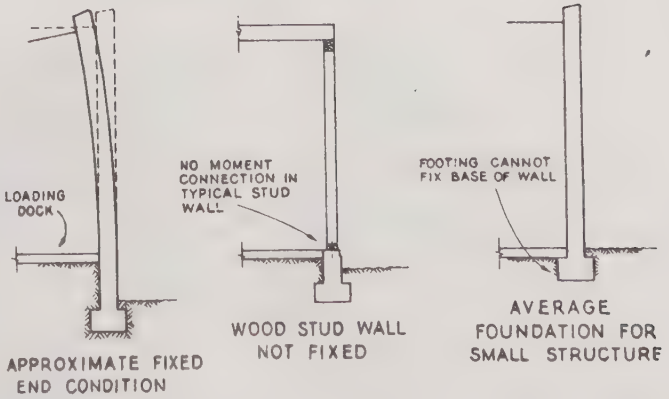
LOADING DOCK
APPROXIMATE FIXED END CONDITION
NO MOMENT CONNECTION IN TYPICAL STUD WALL
WOOD STUD WALL NOT FIXED
FOOTING CANNOT FIX BASE OF WALL
AVERAGE FOUNDATION FOR SMALL STRUCTURE

FIG. No. 15

Earthquake-Resistant Design for Civil Engineering Structures Earth-Structures and Foundations in Japan

By Masanori Numata*
Chairman of Earthquake Engineering Research Committee,
Japan Society of Civil Engineers

Chapter 1. Preface

Unfortunately situated along the circumpacific earthquake zone, Japan has earned fames also for haunting earthquakes of grand scales. The Japanese engineers have been compelled, therefore, to establish a practice to take into account earthquake effects as one of the major external forces to be considered in the design of almost any type of structure. The practice demands various careful treatments especially for such public utilities as earth-structures, foundation works, bridges, dams, harbor facilities and water supply systems;each treatment embodying time-honored experiences plus results of scientific researches of Japanese engineers.

As a token to commemorate the 2nd world conference on earthquake engineering, the following six authorities, namely,
the Japanese Society of Soil Mechanics and Foundation Engineering,
the Bridge and Structural Committee, Japan Society of Civil Engineers,
Japanese National Committee on Large Dams,
Port and Harbor Bureau, Ministry of Transportation,
Japan Waterworks and Sewerage Association and
Earthquake Engineering Research Committee, Japan Society of Civil Engineers
compiled histories of earthquake damages, current aseismic design procedures, important results of research activities and other related data in their fields.

Though every one of the presented data is deemed a contribution of great interest to enriching the aseismic engineering at large, it is not possible to present them at the conference in their entirety in the limited space assigned to each author. The writer will therefore confine himself to trying only a general review of these data. For further details, readers are refered to the recent publication of the Japan Society of Civil Engineers, titled "Earthquake Resistant Design for Civil Engineering Structures, Earth Structures and Foundations in Japan".

Chapter 2. Earth structures and foundations

§1. Current methods of earthquake resistant design

1) Design seismic-coefficient

For the purpose of earthquake-resistant design of foundation for building and civil engineering structures, including earth structures, the so-called seismic-coefficient method is commonly used at present. In the seismic-coefficient method the force equal to the weight of a mass multiplied

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* Professor, School of Science & Engineering, Waseda University, Tokyo, Japan

by the specified value of seismic-coefficient is assumed to act statically at the center of gravity of the mass, and the stability of a structure is checked under the above loading condition. In general, the action of the seismic force is considered in horizontal direction alone, whereas in some cases the vertical force is also taken into account.

Values of design seismic-coefficient specified in different fields of construction engineering will be shown in the followings.

a) Building foundation

Earthquake force is assumed to act in horizontal direction alone and the value of design seismic-coefficient, k, is determined within the specified range of 0.096 - 0.300 depending upon the ground condition and the type of the construction of the building.

b) Bridge

See Chapter 6.

c) Dam

See Chapter 4.

d) Port and harbor structure

See Chapter 5.

e) Embankment

For earth structures such as river dike, coastal levee or highway embankment, no effect of earthquake action is taken into account. The reasons for this treatment are that (1) river dikes and highway embankments may be easily repaired even when they are damaged and (2) furthermore, for river dike which is to be designed with due consideration of flood, it may be permitted to assume that flood and earthquake seldom occur simultaneously.

2) Earth pressure

To compute earth pressure upon a retaining wall under the action of earthquake force, Coulomb's earth pressure formula is used in general, with the assumption that the back-fill is acted upon by horizontal acceleration k x g (g: acceleration of gravity) or both by horizontal and vertical accelerations. However, when conditions are favorable, the increments of earth pressure due to earthquake are sometimes disregarded.

3) Soil reaction

Soil reaction acting on the base of footing under the influence of earthquake forces is usually assumed to be distributed in a trapezoidal or triangular form.

4) Bearing capacity of soil

For building foundations it is permitted during earthquake to double the allowable bearing value of the subsoil or of the pile for normal case. This concept is, however, not always supported in other fields of construction and the ways of estimating bearing capacity under the action of earthquake are quite different.

5) Resistance against sliding

The resistance against the sliding of a structure is also examined when the structure is subjected to earthquake forces.

6) Earth structures

In design computation of earth dam, it is assumed that the earthquake force acts in horizontal direction and the factor of safety is calculated by taking the ratio of total shearing resistance to developed shearing force along the most unfavorable circular surface of sliding. Furthermore the pore water pressure is also taken into account in the above computation.

The angle of slope of the downstream side of a rock-fill dam for slag-dump of an ore mine is determined so that the following condition is satisfied:

$$i < \Phi - \tan^{-1} k$$

wherein

i : angle of slope
Φ : angle of internal friction of rock-fill material (assumed to be equal to angle of repose)
k : design seismic-coefficient

7) Factor of safety

In some fields the factor of safety for a structure is reduced during an earthquake in a similar way to above stated building foundations. This idea is based on the momentary character of earthquake and the economy of construction. Since, however, the effect of earthquake force has not been fully revealed as yet, the reduction of the factor of safety during an earthquake is often disfavored.

For an earth structure such as earth dam, the factor of safety may be equal to unity during an earthquake, which is determined as mentioned before as the ratio of total shearing resistance to shearing force developed along the most unfavorable circular surface of sliding. In general, thus designed earth structure provides the factor of safety equal to 1.5 to 1.7 in normal cases.

The factor of safety during an earthquake as to sliding of gravity-type quaywall may be reduces to unity, while for some railway structures such as retaining walls the factor of safety for earthquake force is often considered as same as in normal case.

§2. Problems in earthquake resistant design

Because of the facts that the effects of earthquake, especially on earth structures and foundations, are not precisely known as yet and that the desingning is inevitably influenced by economy, it may be said that the general policy for earthquake-resistant design in Japan is likely to be underlain by a philosophy of resignation.

The reduction of the factor of safety during an earthquake as stated in the previous paragraph may not necessarily be justified, if a great earthquake loosens the ground or causes an abrupt increase of pore water pressure and a loss of strength of subsoils. However, as to the behavior of the subsoil beneath a structure or the change of pore water pressure due to earthquake, may things remain still unknown.

On the other hand, the full-scale and model tests under static loadings carried out in the National Railways show that some assumptions in conventional theories of stability computations are not adequate. For instance, it has been an accustomed practice to check the stability of a shallow footing for overturning and sliding separately; however, it is shown that the resistance against sliding changes with the magnitude of overturning moment and, consequently, the conventional practice to assume a constant resistance against sliding is not correct. Both effects of overturning and sliding are thus closely related to each other and should not be dealt with separately.

The usual assumption, that soil reaction upon the wall surface of a well or a caisson acts purely in horizontal direction, distributions in a parabolic form and is not allowed to exceed the value of Rankine's passive pressure, is far from the actually observed result. In reality, the reaction at the wall surface is not horizontal but involves a vertical component due to friction and adhesion. Even whan the magnitude of this reaction exceeds far more than Rankine's passive pressure, the soil does not perfectly reach the state of yielding.

Fortunately, such inadequacies in the theory of stability computations are gradually being removed and corrected to some extent by analysing results of static tests; nevertheless, as for deformation characteristics and strength of subsoils under the action of dynamic forces such as earthquakes, it could not be overemphasized that many unsolved, important problems should be studied in the near future.

While various attempts have been made with respect to vibration tests of model foundations, satisfactory conclusions have not yet been obtained.

It is expected that many uncertainties would be made clear if full-scale foundations were subjected to strong, artificial vibration and their behaviors were carefully observed. Unfortunately, however, it is quite difficult in the present state of techniques to generate and control at one's disposal such a strong vibrational motion that would cause damages to the specimen. Consequently, in addition to the development of basic theories, the establishment of empirical earthquake-resistant design method based on analyses of observed earthquake damages is most important under the existing circumstances. In order to promote such an empirical studies the Japanese Society of Soil Mechanics and Foundation Engineering should like to make proposal to World Conference on Earthquake Engineering to establish an international standards of questionaire on earthquake damages. It will

be a great help to have unified and comprehensive information on earthquake damages at every part of the world.

Chapter 3. Water supply works

§1. Introduction

Aseismic structures for the water supply works in our country are designed in accordance with the standards determined by the Japan Waterworks and Sewerage Association in 1953. These standards specify the general principles of aseismic structures and the detailed treatises of special structures for water supply works. Some notes about the latter will be given here.

§2. Detailed treatise

A. Water source installations

(1) Careful precaution shall be exercised so that the outlets and outflow channels, either of open-channel or of tube type, may not be clogged by surface soils and rock debris originating from adjoining hills at the time of earthquake.

(2) Intake tower

(a) The most desirable type of an intake tower is a reinforced concrete structure resembling a cylindrical appearance.

(b) The intake tower shall be given a reasonably deep embedment into a strong foundation. When a sunk-well technique is applied, the embedment depth shall be sufficiently large and the bottom lined with concrete to a considerable thickness.

(c) The conduits leading away from the intake tower shall be, if possible, iron pipes embedded deep below the water bottom.

(d) It is desirable to make a foot bridge approaching the intake tower as light as possible so that the tower may remain free from any extra load.

(e) It shall be avoided to locate the intake tower at such places where there is a possibility of burial due to collapse of river embankment or hill slopes, or of scouring due to currents.

(f) In computing the stability of an intake tower, a particular emphasis shall be placed on the buoyancy so that the safety of the structure may be guaranteed against earthquake shocks when the interior is empty.

(3) For such intakes of gate and pipe type the same sort of care shall be exercised as for the case of a reservoir.

(4) The intake pipe crossing an embankment shall be made of iron, laid upon a strong solid foundation, well protected with concrete, and refilled with special care.

(5) As an infiltration gallery, a perforated reinforced concrete pipe equipped with sockets of sufficient strength and considerable depth shall be laid on a homogeneous foundation: the spigot shall be inserted sufficiently deep into the socket and its periphery protected with strong wooden frames or other devices.

(6) It is desirable that the attached manholes and junction wells shall be placed on an especially strong foundation, and the joints connecting 2 or 3 succeeding pipes immediately adjoining them on both sides shall be dressed in comparatively long thimbles.

(7) The portions of an infiltration gallery other than the infiltration pipes, including the part crossing an embankment, shall be made of high-qu-

ality pipes without openings and the joints shall be completely water tight.

(8) The side walls of a shallow well shall be, as a rule, of a cylindrical form and built of reinforced concrete.

(9) A deep well casing shall be equal or superior to the JIS-G3427-type gas pipe; the joints and strainers shall also possess sufficient strength.

(10) Deep well casings shall project beyond the floor slab of the pump chamber.

(11) It is not advisable to shape the upper portion of a deep well casing similar to that of a shallow well in order to use it also as a pump well.

B. Water conveyance installations

(1) In principle, an open channel and a culvert shall be built of reinforced concrete with the combination of the bottom and side walls behaving as a monolithic unit.

(2) It shall be avoided to place them on or inside the refilled soil. When otherwise impossible, the techniques applied to purification installations shall be employed.

(3) The expansion joints shall be installed at approximately 10 to 20 m intervals; they shall also be used at such places where different geological formations meet, and before and behind a bridge, weir, manhole and gate, etc.

(4) The route of a tunnel shall avoid such places as a hill slope which exerts a biased load.

(5) Since the entrance and exit of a tunnel are liable to collapse, they shall be protected adequately.

(6) At such places where there is a fault line runing in the vertical direction or where sharply differring geological formations meet, the tunnel lining shall be insulated and the lining before and behind this position particularly strengthened.

(7) Since an aqueduct structure is top-heavy, the design shall be so arranged as to prevent it from falling off the bearing plate due to a horizontal thrust of an earthquake. For such purposes it is advisable to extend arms from bridge piers or bearing posts, and connect the bridge piers or bearing posts to the aqueduct beams with diagonal members so that both tensile and compressive stresses may be transferred directly to the piers, bearing posts or beams.

(8) The piers or bearing posts of an aqueduct bridge shall be equipped with appropriate transversal straddling post to prevent overturning due to earthquake.

(9) The post of the aqueduct bridge pier and the foundation shall be connected adequately.

(10) The abutment(of an aqueduct bridge), which is generally vulnerable to earthquake shocks, shall rest on a firm solid foundation with a large embedment depth.

(11) Since the connection of the conduit and the aqueduct constitutes one of the most vulnerable points, special care shall be exercised to make it a perfect earthquake-proof structure, or otherwise an efficient expansion joint shall be installed at this position. In cases where an intensive earthquake shock is expected, a further safety shall be guarantteed by installing two joints at a relatively close interval.

C. Installations for purification and distribution

(1) A structure of this category shall not be placed upon a refilled earth or near a slope toe which is apt to collapse. If otherwise impossible, it shall rest on a pile foundation or reinforced concrete columns, either supported on the original ground, and the bearing capacity of the refilled earth shall be disregarded in the process of compution.

(2) For the sloping walls of a sedimentation basin which are inevitably built on an artificial fill, the soil shall be adequately compacted and the construction shall start only after an adequate amount of settlement and hardening of the fill has taken place. In the regions where the value of seismic coefficient exceeds 0.3, the structure shall on no account be placed on an artificial fill.

(3) The tops of the foundation piles shall penetrate into the structure as deeply as possible (30 to 50 cm, if possible).

(4) It is not advisable to construct a unit structure astride a composite foundation which consists of layers with different bearing capacities. If otherwise unavoidable, it shall be divided into separate units in proportion to the capacities of the supporting layers.

(5) If it is necessary to place a unit structures astride a composite foundation, the layer of lower capacity shall be improved by pile driving or other treatments to equalize the bearing capacities of the entire layers before erecting the structure, and more-over, expansion joints shall be installed at the boundaries of the neighboring structures.

(6) Any type of structure having simpler appearance both in plan and profile and devoid of abrupt changes in configuration is advantageous in resisting earthquake shocks. Accordingly a large haunch shall be placed at the rigid joints.

(7) Particular care shall be exercised in designing the connecting points of different structures since they are vulnerable to earthquake shocks.

(8) For better resistance against earthquake shocks a circular plan shape is generally preferable as a basin-well structure to an angular one.

(9) The floor and side wall of the basin-well shall be built of reinforced concrete in accordance with the earthquake- proof design and rest on a solid foundation so that their combination may act as monolithic unit.

(10) The side wall and floor of the basin-well shall be made watertight by applying high-quality concrete elaborately. In case a water-proof motar coating is applied to a purified water storage basin or a basin of a large depth, it shall be reinforced with thin bars arranged densely both in longitudinal and latitudinal directions. The same precaution will also improve the safety of the protection concrete or moter coating for a waterproof layer of asphalt or other materials.

(11) On the side walls of the basin well the expansion joints shall be placed near the corners. At other portions the expansion joints shall be inserted at intervals of 6 to 9 m for a thin wall and 15 to 18 m for a thick wall (10 m ordinarily and 15 m in maximum). The gap of the joint shall be ordinarily 1 to 3 cm.

(12) A canopied basin shall be desinged in such a manner as to allow safe entrance and exit even at the time of earthquake, and hence every arrangement shall be provided to satisfy this condition.

(13) Since the doorways and inspection windows are vulnerable to local damages due to earthquake they shall be reinforced with special care.

(14) The foundation of a water tower shall rest on a ground of particular reliability. It is desirable even on a favorable ground to secure as great an embedment as possible.

(15) A cylindrical structure of a water tower is advantageous to safety against earthquake.

(16) The riser portion of an intake pipe which is housed inside the water tower shall be supported along the wall surface. It shall be born, if available, by the columns in the center which support a canopy for a large tower. It is advisable to connect the columns with horizontal diagonals.

(17) If further the inside face of the water tower wall is made watertight by lining with steel plates more than 3 mm thick, a fear of leakage will be much reduced assuring extreme safety against earthquake shocks.

(18) The material for an elevated tank shall be steel in preference to reinforced concrete in view of safety against earthquake shocks.

(19) The riser portion of various types of attached pipes to an elevated tank shall be equipped with earthquake-proof joints near the ground surface.

(20) The elevated tank shall be provided with as many supporting columns as possible, with horizontal and diagonal members adequately arranged to prevent buckling.

(21) The tank shall be tied fast to the supporting frames.

(22) A reclining type is recommendable as safe for a pressure tank.

D. Pipeline and appurtenances

(1) The most preferable earthquake-proof method is to select a reliable ground for the site of a pipeline route.

(2) The route shall avoid an abrupt bend either horizontal or vertical. When an abrupt bend is included due to unavoidable circumstance, it is necessary for safety to place a supporting platform equipped with earthquake-proof joints before and behind it.

(3) It shall be avoided to lay a pipeline on refilled earth. If otherwise unavoidable, the technique specified for purification installations shall be employed.

(4) The embedment-depth of pipeline shall be arranged in such a manner as to allow convenient maintenance as well as easy repair.

(5) A solid uniform ground is desirable along entire route. A soft ground shall be adequately reinforced by pile driving, ladder struts or concrete foundation even for a small pipeline.

(6) At such places where soft and firm grounds join, an earthquake-proof joint shall be installed near the latter.

(7) At such places along the trunk route where the specified design coefficient is more than 0.3, it is safe to install expansion joints at the rate of one for every 3 joints.

(8) It shall be avoided to lay the trunk route of a pipeline on the reclaimed area, vicinity of ditches, river banks, beaches, cliffs or bulkheads.

(9) At such places where the route crosses over a sewage pipe or other establishments, a gap shall be maintained between them.

(10) The distribution pipes shall not have a dead end; the pipe terminals shall be connected to each other.

(11) The distribution pipelines shall all be equipped with gate valves

installed in such a manner that in event of damages inflicted on any part of the system the suspension of water supply operation could be confined to as small a region as possible.

(12) Even such types of pipe as a delivery pipe and a trunk distribution main pipe which do not possess branch pipes shall be equipped with sluice valves at an interval of approximately 1000m.

(13) It shall be avoided to lay the specials continuously. A cut or straight pipe shall be inserted between 2 adjoining specials.

(14) Near the position of a tee or cross pipe along a key route it is desirable to install earthquake-proof joints at the two pipelines meeting at right angle.

(15) When inserting along the pipeline such appurtenances as a fire hydrant or sluice valve, etc., which present different characteristics of vibration due to earthquake from that of the pipe, it is desirable to install earthquake-proof joints before and behind it.

(16) The bends, dead ends or tee pipes shall be protected by placing a concrete platform in direct contact to the outer surface of them or driving reinforced concrete piles. It shall be avoided, however, to install the bends inside a concrete block.

(17) Various types of pipes which penetrate through the surrounding wall or bottom slab shall not touch the wall body directly. When it is necessary to install them in direct contact to the wall in order to ensure water-tightness, an earthquake-proof joint shall be installed apart from but near the wall. The same procedure is recommendable between a pump and a suction or pressure-conveying pipe.

(18) A suction pipe which is suspended down a pump well shall be fixed to the wall with strong metal fittings to prevent it from vibration in a different manner from the wall.

(19) A pipeline running on the ground surface for a great length shall be equipped with a fixing platform at an interval of approximately 300 m to fix the pipeline to the subsoil (namely, fixing with iron bars to a dented platform), together with expansion joints set between them.

(20) In case a high-quality cast-iron pipe is used and unless an effective earthquake-proof joint is employed a lead socket joint shall be used as a rule.

(21) Special care shall be exercised in treating the upper and lower portions of the perimeter of a lead connection.

(22) When using a steel pipe, a flange or electric-welded joint beside a socket lead joint is recommendable; and the former two types of joint shall have an expansion joint inserted at an interval approximately 30 to 60 m.

(23) A siphon crossing shall not be built on unfavorable ground conditions.

(24) A flexible joint durable to water pressure shall be used for a siphon crossing.

(25) The approach pipes before and behind a siphon crossing shall have as gentle a curvature as possible, and any curve shall be fixed sufficiently with a concrete platform.

(26) Sluice valves shall be palced at both ends of a siphon crossing.

(27) A siphon crossing of a large-calibered pipe or of a key trunk line may be divided into 2 or more branches for safety.

(28) Unless otherwise unavoidable, a pipeline shall not be laid on a wooden bridge.

(29) A pipe bridge shall be located away from an existing wooden bridge by at least more than 2 m upstreamside.

(30) A pipe bridge shall be built of fire-proof material, and on a soft foundation a steel plate-girder or a steel frame girder is recommended for safety.

(31) It is advantageous against earthquake shocks to build the superstructure as continuous system on relatively favorable ground.

(32) The superstructure shall be adequately anchored on the substructure with anchor-bolts in order to prevent it from falling off the position at the time of earthquake.

(33) It is safe to use a steel pipe for a bend pipe or an S-pipe which connects to a bridge-born pipe and an underground straight pipe behind the abutment. It is advisable to treat the joints of the steel pipe by electric welding.

(34) A bridge-born pipe shall be fixed to the superstructure at each span, and also equipped with an expansion joint at each span.

(35) For a bridge-born pipe made of cast-iron a flange joint shall be avoided.

(36) At the position where connection is made to the bend pipe on either end of a bridge-born pipe an expansion joint shall be placed without exception.

(37) A pipe mounting to the abutment shall be sloped by less than 45° and connected tightly to a straight pipe before or behind the bridge.

(38) The both ends of a bridge-born pipe shall not be fixed to the abutments, but allow for a considerable gap around the pipe.

(39) Special care shall be exercised to refilling the banking behind the abutment, and a foundation supporting a bend pipe shall be particularly reinforced by pile driving.

(40) A pipe bridge having the steel pipe itself as the main girder shall be a simple beam structure especially on a soft foundation and equipped with flexible joints on the abutments and piers.

(41) Gate valves shall be installed at the pipeline before and behind bridge.

(42) A gate valve shall not be placed close to a building or any other establishment in order to ensure convenient and rapid replacing and repairing.

(43) It is desirable to insert an expansion joint before and behind a gate valve.

(44) Sluice valve chamber or fire hydrant chamber shall be built of reinforced concrete or, preferably blocks of concrete or brick, so that damage repair may be convinient.

(45) It shall be avoided to use a gas pipe as a service pipe, since its screws are vulnerable to an earthquake shock.

(46) When a steel or copper pipe is used as a service pipe, a lead pipe approximately 500 mm long shall be inserted before and behind the connection to a ferrule, stop valve or hydrometer. The connection to a large tap post such as an anti-freeze common tap shall also comply with the same provision.

E. Power and pump

(1) An extra power unit of, if possible, different type shall be maintained for emergency use.
(2) When only the electric power is available, an extra unit with different source and supply route shall be provided for emergency use.
(3) The delivery end of a pump shall be directed horizontally or downward.

Chapter 4. Dams

§1. Damages to dams in past earthquakes

In Japan, from ancient days agriculture developed primarily with the cultivation of rice paddy fields. To assure supplies of water for irrigation purpose, many earth dams have been built and among the existing dams, the oldest date back to over 1,000 years. These dams were constructed based on experiences during those days. Most of the dams, approximately 50 meters in height, including other purpose dams were constructed in the past thirty years. Therefore, at the time of the 1923 Kanto Earthquake the number of high dams in existence was extremely small. Since then, Japan has been affected by several severe earthquakes. Investigations of damages to dams caused by these earthquake were important factors in the researches on earthquake resistant features in the design of dams.

In this investigation the method followed was to circulate questionnaire forms to administrators of dams. However, it was technically difficult to investigate all phases of damages. Therefore, investigations were made of the 1923 Kanto Earthquake and the 1944 Nankai Earthquake which caused comparatively heavy damages to dams. In both earthquakes, as a principle, investigations of damages were made on high dams selected in areas where the intensity exceeded scale V(80-250 gals) established by the Central Meteorogical Observatory.

(1) Damages to dams in the Kanto Earthquake

A description of the dams that were damaged by the Kanto Earthquake are given below:

(A) Ōno dam

The Ōno Dam is located in Ōno, Uenohara Town, Kita-Tsurugun in Yamanashi Prefecture. The dam constructed on the Yata River, a tributary of the Katsura River, in the Sagami River System is an earth dam with a center impervious core. The height of the dam is 37.3 m above foundation and 49.1 m from the base of the cut-off wall. The center core is concrete and soil concrete. The dam body is red earth which was placedin lifts and compacted. The dam was completed in 1914 and nine years later it was hit by the Kanto Earthquake.

The dam axis runs generally in the direction of N 21° E. The left abutment curves towards the upstream. The geology of the right bank is palaeozoic strata and left bank is alluvium. The major part of the dam rests on bedrock.

From observations of toppled stone monuments in the locality of the dam, it was estimated that the earthquake intensity was about 330 gals or Intensity Scale VI. Settlement of about 3 cm took place in the center of the dam crest and about 25 cm at the curved section of the left abutment. A crack almost 20 cm wide, 6 m deep and 40 m long appeared on the crest near the left bank. On the upstream slope several cracks appeared running parallel with the dam axis, but the cracks were in the riprap and did not extend into the dam body. Several cracks appeared also on the down stream slope within the distance of about 10 m from the crest. The crack nearest to the crest was about 11 m deep and 30 m long was observed and numerous small cracks in the vicinity of the drain duct in the berm. Swelling on the slope about 18 m below the crest was noticed.

(B) Murayama-kami Dam

Murayama-Kami Dam is located in Imokubo, the Town of Yamato, Kitatama-gun in Tokyo. This is an earth dam 24 m high with a center core. The direction of the dam axis is N 10° E. This dam was completed in June 1923, just immediately before the Kanto Earthquake. The geology of the dam site is loam formation of the Tertiary Period. The core puddle is a blend of clay and sand mixed to a ratio of 2:1. The material was placed in layers of about 9 cm in thickness and compacted with rollers to 6 cm. In the other sections of the dam, earth was placed in layers of about 15 cm in thickness and compacted to about 9 cm. In the upstream slope of the dam, a layer of gravel 30 cm in thickness was laid on which concrete blocks 30 to 45 cm thick and 1.8 m square were placed. The void between the concrete blocks were filled with clay.

The earthquake intensity in this district was estimated to be in the scale of VI. The reservoir pool was 7.58 m below full water level and the effective drawdown was 5.94 m. Damages by the earthquake were movement of the concrete block facing by approximately 6 cm, destruction of a drain duct and parapet wall erected on the dam crest. Settlement of about 18 cm was observed near the center on the crest. On the downstream slope, settlement of about 1.2 m was observed near the center and a crack about 12 cm wide and 110 m long appeared in the downstream slope protection. Settlement of about 1.2 m was observed around the center of the drain ducts in the berm and in the downstream foot of the dam. Generally speaking, deflection was observed on the horizontal plane. However, cracking did not happen in the slope of the dam body.

(C) Murayama-shimo Dam

Murayama-shimo Dam is located near Murayama-kami Dam. The construction method applied to Murayama-kami Dam was followed at this dam. At the time of the 1923 Kanto Earthquake the dam had reached a height of 16.2 m of a total height of 30.3 m. Three cracks running parallel with the dam axis appeared approximately 25 m upstream of the dam center. One of these cracks was about 2.4 cm wide, 10 m deep and 68 m long. It was recognized that the foundation where this crack appeared was the weakest zone.

(2) Damages to dams by the Nankai Earthquake

There were 7 instances in which high dams were damaged in the Nankai Earthquake. All of those dams were for irrigation purpose and aside from the Honen-ike Dam in Kagawa Prefecture, which is a multiple-arch concrete dam, the rest are earth dams. Of the dams damaged in that earthquake, descriptions are given of the Otani-ike and Honen-ike Dams.

(A) Otani-ike Dam

Otani-ike Dam located in Otani, Komatsu Town, Ehime Prefecture is an earth dam with a center impervious core. This dam is 27 m high, 8 m wide at the crest, 116.4 m wide at the base, the slopes of the upstream 1:4 and the downstream 1:3, and the total embankment 168,800 m^2. This dam was completed in 1920 and 26 years had lapsed when it was struck by the earthquake. The foundation of the dam is sandstone, the direction of the dam axis is N 85°E and the depth of the water in the reservoir at the time of the earthquake was 16 m.

The earthquake caused a crack on the crest approximately 84 m long and running parallel with the dam axis. In addition, cracks running parallel with the dam axis appeared on the upstream slope at distances of 12 m and 22 m from the upstream end of the crest. These cracks were respectively about 60 m and 24 m long. The culverts in the base of the dam and in the left abutment were damaged. In repairing the dam, earth was compacted where the cracks occured and cement was grouted from the culvert.

(B) Honen-ike Dam

The Honen-ike Dam is located in the village of Gogo, Mitoya-gun, Kagawa Prefecture. Minor damages were suffered by this dam in the Nankai Earthquake. The dam is 30 m high and the length at the crest is 144 m of which the center 87 m is a multiple arch structure adjoined to concrete gravity structures on both ends. Each arch section with a span of 14.4 m, is 5.45 m thick at the crest and 8 m thick at the foundation.

The earthquake caused a crack in the center arch section near the joint at the buttress and leakage of water was observed. Leakage occured also in the left abutment where the foundation rock (Izumi sandstone) is weak. The damages were repaired by grounting cement.

(3) Damages to dams by earthquakes

From the foregoing discussion, it is possible to draw a general view on earthquake intensity which have caused damages to various types of high dams. Maximum earthquake intensity experienced with concrete gravity dams was within the scale of VI (250-400 gals) and no damages have happened. However, there have been a few cases in which cracks appeared in the base of overflow gate piers.

Earth dams were damaged by earthquakes of an intensity scale over IV (25-80 gals), and the damages experienced are cracking, sliding of the slopes and collapsing. The degree of damage cannot necessarily be determined by the seismic intensity. Though the intensity may be the same, the degree of damage is dependent on the motion of the earthquake. But, it is recog-

nized that the seriousness of damages is greater to a dam near the epicenter than away from it. The degree of damages to an earth dam greatly depends on the physical properties of embankment materials and the appropriateness of the construction practice, and that a number of the dams reported in this investigation were constructed in an era before modern construction techniques were developed.

Arch. hollow gravity and rock-fill dams are types of structures which have developed comparatively in recent years and have not experienced serious earthquakes. Their ability to withstand earthquake shocks remains to be tested.

§2. Earthquake resistant design methods for dams

Earthquake resistant calculation methods generally followed in the design of dams is the so-called earthquake intensity method in which the weights of the dam and a part of stored water determined by the formula of dynamic water pressure, are multiplied by the seismic intensity and these forces of inertia are applied horizontally on the dam body to calculate stress and stability. This method has been in practice in Japan from the time high dams began to be constructed. Since then as years have passed improvements to the method have been made in substance, and with developments in the researches and studies of earthquake phenomenon and earthquake resistant properties of dams, at present the earthquake intensity taken into account in the design of dam is determined by various factors such as the type of dam, foundation rock, occurence of earthquake in the past in the area where the dam is to be constructed, and the magnitude of influences on the downstream reaches. In the case of an important dam, model tests are made to study vibration features, equivalent seismic coefficient are considered dynamically, and measuring of stresses under assumed earthquake conditions are considered dynamically, and measuring of stresses under assumed earthquake conditions are conducted to examine closely the stability of a dam in an earthquake.

Table 1 is the design seismic coefficient given in the design criterion for dams established in 1957 by the Japanese National Committee on Large Dams and this criterion is commonly followed in Japan today. Half the values shown in Table 1 may be taken when the reservoir is empty because in case of an emergency the damages caused thereby not be serious.

Table 1. Design Seismic Coefficient

Type of dam	District	
	Tohoku region(Fukushima, Akita, Miyagi Prefectures); Kanto region: Chubu region; Kinki region; Southern Shikoku region	Hokkaido region; Hokuriku region; Tohoku region(Iwate, Yamagata, Aomori Prefecture); Northern Shikoku region; Kyushu region
Concrete dams and rock-fill dams	0.12 - 0.20	0.10 - 0.15
Earth dams	0.15 - 0.25	0.12 - 0.20

In the case of a concrete dam, on the stress obtained from the above seismic coefficient, the allowable stress under static load condition only is increased by 15 to 20 %.

In the following paragraphs are described examples of earthquake resistant designs applied in existing dams.

(1) Ogochi Dam (concrete gravity type)

The Ogochi Dam is a non-overflow concrete straight gravity structure constructed on the upper reaches of the Tama River in the Kanto region. The dam is 149 m high, 353 m long at the crest and the volume content of concrete is 1,675,680 m^3. This dam was built by the block system and keys were provided in both the longitudinal and transverese joints which were bonded by grouting after cooling of the mass concrete.

Stress calculations made in the design of the dam were both gravity and trial load twist methods of analyses which were conducted under the following combinations of loads.

(a) static water pressure at full water level-silt pressure-earthquake force in the downstream direction-dynamic water pressure-dead load -up-lift pressure.
(b) static water pressure at full water level-silt pressure-dead load -up-lift pressure.
(c) dead load-earthquake force in the upstream direction.

In the trial load analysis, variation of stress conditions in ungrouted and grouted transverse joints and other conditions were analysed.

The equivalent horizontal seismic coefficient taken for full reservoir condition was 0.12 and that for empty reservoir 0.06. The allowable compressive stress of concrete under earthquake condition was taken at 46 kg/cm^2. To study the results of the design calculations, model tests were made by the photoeleastic method. However, in this analysis, the earthquake force was considered as static external force. Studies are being continued on the behavior of the dam to vibration caused by natural earthquakes.

(2) Ikawa Dam (hollow gravity type)

The Ikawa Dam is a hollow gravity type concrete structure, 103.6 m high and 243 m long at the crest, constructed on the upper reaches of the Oi River in the Chubu region. The upstream and downstream faces are 1:0.55 and the section is I shaped.

Earthquake resistance studies in the design of this dam were for the following load combinations.

(a) static water pressure at full water level-silt pressure-earthquake force in the downstream direction-dynamic water pressure-dead load.
(b) earthquake force in the upstream direction-dead load.
(c) earthquake force in the direction of the dam axis-dead load.

The equivalent horizontal seismic coefficient taken for full water condition was 0.12 and that for empty reservoir 0.06. For stress calcula-

tions, in the case of (a) and (b) load conditions, load distribution was assumed to be trapezoidal and in the case of (c), buttress wall was assumed to be fixed trianglar slab. Stress distribution was studied by three dimensional photoelastic experiments to check and correct the computed values.

From the standpoint of earthquake resistance, case (c) was considered to be the most important one because of the existence of tensile stress, though very little, and vibration experiments with rubber models were supplemented to determine the period and mode of natural vibration. The results of this experiment generally agreed with the calculated values and the primary vibration period of the highest section in the actual dam was estimated to be in the order of 0.11 to o.12 seconds.

After the dam was completed, vibration experiments were conducted with vibrators and records are maintained on the vibration of the dam and foundation caused by natural earthquakes.

The vibration experiments were primarily conducted to study the vibration characteristics in the direction of the dam axis. The vibration period is generally close to the value obtained in the model experiments and the damping constant showed an extremely small value of about 2%. In view of the fact that the fundamental period of the dam is short in comparison with the predominant period of the earthquake motion and consequently, the degree of resonance being rather small, it was concluded that the dam is stable against earthquakes. Vibration records of natural earthquakes are generally in agreement with the estimated values.

(3) Kamishiiba Dam (concrete arch dam)

The Kamishiiba Dam constructed on the upper reaches of the Mimi River in the island of Kyushu is a constant angle arch dam. The standard radius at the crest is 142 m, the center angle 115 , the crest length 330m, the height 110m, the maximum width 27.7 m and the minimum width 7 m.

Earthquake resistant features studied in the design of this dam are stress conditions under the following load combinations.

(a) static water pressure at full reservoir-silt pressure-declineof temperature-earthquake force in the downstream direction- dynamic water pressure-dead load-up-lift pressure
(b) static water pressure at low water level-rise of temperature-earthquake force in the upstream direction-dead load
(c) earthquake force in the tangential direction at arch crown
(d) relative displacement of both banks caused by the phase difference of seismic waves at either banks

The equivalent horizontal seismic coefficient used in the calculations were o.12 at full reservoir and 0.06 at low water level conditions.

Stress calculations were checked by the trial load analysis method and adjustments were made in the radial direction or in the radial direction, tangential direction and rotation.

The maximum stress was found to be in load condition (a) and in the other cases the stress was of a value much smaller and not comparable with (a).

In the case of load condition (a), when the tensile stress exceeded 5 kg/cm^2 on the cantilever elements, recalculations were made as cracking of concrete was assumed to have occured. Allowable compressive strength of concrete was so determined that the safety factor is 5 based on the strength of test pieces (15 cm diameter, 30 cm long) 91 days and the allowable strength of concrete was increased by 15% under earthquake conditions.

In addition to the above described earthquake considerations, in the design of the dam, studies of the vibration features by model tests, observations of earthquakes at the damsite and detailed investigations of acceleration spectrum due to seismic wave action and seismic activity in the vicinity of the damsite were conducted.

The mode and the natural period of the vibration of the dam at empty reservoir condition were studied with models made of agar-agar and those at full and empty reservoir conditions were studied on a rubber model whose specific density coincided with that of concrete. Through these experiments, the modes and periods of the vibrations of the first, second and third order were determined and further the prolongation of the natural period by the storing of water were ascertained.

For the earthquake characteristic in this region, it was found that the predominant period at the damsite was shorter when the earthquake originated in the inland and the epicenter was close, and in the case of an earthquake originating in the ocean some distance away, the period was quite similar to the dam's lowest natural period of 0.3 to 0.4 second. From the acceleration spectrum calculated from earthquake records, it was found that the equivalent seismic coefficient is 3 to 1.5 times the intensity at the foundation for a period of 0.1 to 0.4 second taking a damping constant of 0.1.

On the earthquake activities in the region, based on earthquake records of almost 50 tremors in the Kyushu region, it was estimated that the maximum intensity was in the scale of III to IV. It was also assumed that the influence would be extremely little to this damsite by inland earthquakes of a volcanic nature.

As a result of these stability calculations and experimental studies, it is believed that the stability of the Kamishiiba Dam against earthquakes has been generally satisfied. Since completion of the dam, vibrations caused by natural earthquakes and with vibrators are being studied.

(4) Miboro Dam (rock-fill dam)

The Miboro Dam is a rock-fill type of structure with an inclined impervious clay core under construction on the upper reaches of the Sho River in the Hokuriku region. The dam is 131 m high, 405 m long at the crest and the total embankment is 7,950,000 m^3. Design calculations were made for the following load conditions.

Stability calculations against aliding were made of the dam body (downstream filter zone and rock zone) under full reservoir condition. Stability against bearing strength under full reservoir condition were examined at the base vertically below the downstream end of the crest of the

standard section of the dam. On the stability of the upstream slope, calculations were made by the slip circle method under the following conditionds: at completion of the dam, full reservoir level and low water reservoir level. Stability calculations of the downstream slope was also by the slip circle method. For the foundations, the stability against horizontal shearing force was calculated under empty reservoir and full reservoir conditions. In all of the stability studies described above, calculations were made taking into account earthquake influences as well as not taking into account such influences.

Of the load conditions that were taken into account in the design calculations, the horizontal seismic coefficient was $K_h = 0.12$ and in case of a temporary condition only one half of this value was considered.

As a result of these calculations, the minimum safety factor obtained for sliding was 1.1 and for stability of the slopes was 1.2.

(5) Makio Dam (rock-fill dam)

The Makio Dam under construction on the tributary of the Kiso River in Nagano Prefecture will store water for the Aichi Irrigation System. This dam is a rock-fill structure with a center clay core. The dam is 106 m high from foundation rock, 264 m long at the crest and the total embankment is 2,500,000 m^3. Aminimum safety factor of 1.37 was obtained as a result of studies made by the modified Fellenious' method for various load conditions. A horizontal seismic coefficient of 0.15 was taken in the design calculations taking into consideration earthquake records taken at the dam site and the results of vibration experiments on a model. Acceleration meters and displacement meters are embedded in the dam and observations of earthquakes and measurements of the movement of the dam are being taken.

(6) Ojiya Dam (earth dam)

The Ojiya Dam is a center core, rolled-fill earth structure located at the downstream of the Shinano River in the Hokuriku region. It is 954 m long at the crest, 18.8 m in maximum height above ground level.and 29.3 m above Tertiary bed rock.

Calculations of the stability of the dam were carried out as follows: The stability of the dam to shear force was studied at the base and on the surface of the Tertiary bed of the highest section under normal and earthquake conditions. At the same time the stability of the slopes of the dam under normal condition was examined by the slip circle method when rapid draw-down takes place on the upstream side and when there is a snow load of 1.75 t/m^2 working on the downstream slope and its seepage pressure is at full work.

As a result of the studies, minimum safety factors of 1.18 at the base of the dam, 1.04 on the surface of the Tertiary bed in time of earthquake shock and 1.48 on the upstream slope and 1.47 on the downstream slope under normal condition were obtained. The angles of internal friction of the soil used in the calculation were determined by laboratory shear tests. A seismic coefficient $K_h = 0.1$ was used as an earthquake force and the angle of internal friction was reduced by applying Dr. Sano's method. As the unit weight of earth below the seepage line, saturated unit weight when it

acted as external force and bouyed unit weight when it acted as resistance were adopted. The excessive pore pressure in the clay core taken in the calculation of the slope stability was 1.25 times the vertical height of the core from the point under consideration.

§3. Description on research activities

The history of research activities related to earthquake resistant feature of dams can be braodly divided into two periods. The first period is the years from the Kanto Earthquake to the termination of the Second World War. Research activities during these years in respect of earth dams was treated dynamically and of gravity dams the research was mainly directed to practical calculation methods by the earthquake intensity method. On arch dams there were no studies worthy of mention. This trend in research activities is believed to have resulted from the belief that, in view of actual damages to dams by earthquakes, dynamical clarification of an earth dam was thought essential but as for gravity dams (the highest in existence during that time was the Miura Dam which is only 84.1 m) it was recognized that they were in practicality considered rigid structures.

The second period of research activities is the year from the termination of the War to date. During this period theories of considering earthquake force as dynamic external force and researches by model experiments have developed. These activities were stimulated by definitive schedules of construction of gravity dams, arch dams and hollow gravity dams in the magnitude of 150 m in height, and dynamical researches advance greatly as it was realized that statical researches only were inadequate in the studies of earthquake resistant features of these dams. Whereas researches up to now, in a way, have dealt with the fundamental problems of vibration, in recent years the practical application of theories have been pursued. In order to obtain necessary date for this purpose, measurements of dynamic characteristics and vibration during earthquakes in existing dams and researches in the dynamic strength of structures and structural materials (concrete and earth) have been stated.

Generally speaking, the researches on the earthquake resistant features of structures have been made to understand the properties of earthquake motion expected in the region where a structure is to be built and the properties of structures that can resist such forced vibration and to find an appropriate earthquake resistant design method. It will be noted from the general description of earthquake resistant studies of dams in Japan given in the foregoing chapter our knowledge with respect to the earthquake motion at a damsite and the dynamical properties of a dam are by no means sufficient. The trend of abstractness in many of the studies is believed to be due to the inadequacy in the accumlation of concrete knowledge. Fortunately, seismometers and other measuring instruments have been installed in 17 important dams recently constructed. There are plans to maintain records of earthquake motions of the dams and foundations at the Japanese National Committee on Large Dams.

In the design of important dams dynamic studies by model tests are being conducted to estimate their earthquake resistant properties. Exciting machines and large shaking tables for model experiments essential for the above studies are installed in the Technical Research Laboratory of the Cen-

tral Research Institute of Electric Power Industry, Institute of Industrial Science, University of Tokyo and Engineering Research Institute of Kyoto University. On the dynamic properties of materials used in the construction of dams, the studies are being performed to find the non-elastic properties of the materials going a step forwards from their elastic properties.

Chapter 5. Harbor structures

§1. Fundamental items in aseismic design of quay walls

1) Design seismic coefficient

Current aseismic design of quay walls are briefly reviewed in this paper following the newly revised "Japan Harbor Engineers' Manual (1959)". As shown in the manual, in the aseismic design the whole effects of an earthquake are substituted by static force obtained by multiplying seismic coefficient to the mass in question. Consequently proper estimation of seismic coefficient K, which is the ratio of seismic force to gravity, is essential to an aseismic design. Although many factors should be considered in estimating design K-value, it is done empirically. As to locality of proposed harbor structure, K-values being different for each region of this country is given in the manual by refering to a study of Dr. Kawasumi on the regional probability of occurence of destructive earthquakes. It is also to be noted that in the current design only the horizontal seismic coefficient is taken into consideration.

Seismic coefficient in the air should be increased in the water due to buoyancy. This increased coefficient is called apparent seismic coefficient and it is given by the following equation.

$$K' = \frac{\gamma}{\gamma - 1} K$$

where K' : Apparent seismic coefficient in the water
K : Seismic coefficient in the air
γ : Unit weight of the mass in the air (for the soil it should include the weight of water saturating the soil mass)

2) Lateral earthpressure in an earthquake

Mononobe-Okabe formula is used in the computation of lateral earthpressure in an earthquake.

$$P = \left\{ \Sigma \gamma h + \frac{\gamma \cos \Phi}{\cos(\psi - \beta)} \right\} C$$

$$C = \frac{\cos^2(\Phi \mp \psi - \theta)}{\cos\theta \cos^2\psi \cos(\delta + \psi \mp \theta) \left[1 \pm \sqrt{\frac{\sin(\Phi \pm \theta)\sin(\Phi \mp \beta - \varepsilon)}{\cos(\delta + \psi \pm \theta)\cos(\psi - \beta)}} \right]^2}$$

where Φ: angle of internal friction of backfill,
δ: friction angle between wall and soil,
ψ: inclined angle of back surface of wall to the vertical,
β: angle of ground surface of backfill to the horizontal,
h: height of the wall, and θ : $\tan^{-1}K$

In this equation, following values are usually to be used. The unit weight

of sandy soil is 1.8 t/m³ above the water level in the backfill and 1.0 t/m³ below the water level in the backfill, and angle of internal friction of sand is 30° for general case and 40° for particular good condition, and friction angle between wall and soil is 15° - 20°.

Correction should be made for the lateral earthpressure below sea water level as follows. Lateral earthpressure at the water level in the backfill and that at the bottom of the wall are computed by employing seismic coefficients in the air and that in the water respectively and straight line connecting these two values of lateral earthpressure gives the lateral earthpressure distribution under water.

3) Dynamic pressure of water in an earthquake

The dynamic pressure of water in the backfill is not considered in the current design procedure, because during an earthquake the water might move together with soil particles and dynamic water pressure is to be included in the lateral earthpressure when it is computed by employing apparent seismic coefficient. Also the dynamic pressure of water in front of the wall is not considered because of the complexity of its characteristics such as difference of phase from that of wall movement of lateral earthpressure.

4) Bearing capacity in an earthquake

In addition to the value for the static,state, a great increase of inclination and eccentricity of load should be taken into consideration during an earthquake. The circular sliding surface method is employed in the computation of the bearing capacity during an earthquake.

5) Lateral resistance of piles

For the lateral resistance of a vertical pile the following values are standard in design

for clayey soil	····· < 5 ton
for clayey soil when replaced the top 2 m layer with dence sand	····· < 7 ton
for sandy soil	····· < 10 ton

The lateral resistance of group of piles is considered to be the total sum of lateral resistance of single piles.

Lateral resistance of coupled piles is computed from two axial forces each of which might take the ultimate bearing capacity of pile. The estimation of the bearing capacity is made by (1) loading test of pile, (2) statical bearing capacity formula, or (3) pile-driving formula. The safety factors of 2.5 are used for the cases (1) and (2) and 6-7 for the case (3).

6) Stability of slopes during an earthquake

The stability of slopes in an earthquake is analyzed by the circular sliding surface method, by taking horizontal seismic force into consideration. The lower limits of safety factors in the static and seismic conditions is to be 1.5 and 1.2 respectively when a permanent structure is proposed. These values are reduced to 1.2 and 1.0 in the case of a temporary or an unimportant structure. Usually in sandy soil or gravely soil the base

failure need not be considered. However, the possibility of the toe failure should carefully be examined.

§2. Aseismic design of quay walls

1) Gravity type quay walls

Main external forces acting on the wall in an earthquake are lateral earthpressure, dynamic water pressure and mass force of the wall itself. The computation should be done on three possible causes of failure, namely on (1) the sliding of the wall along its base, (2) the high toe pressure in excess of bearing capacity of the foundation soil and on (3) the sliding below the foundation. The safety against sliding along the base will be secured by making the ratio between horizontal and vertical components of the total external force smaller than coefficient of friction along the base. The safety factors are to be 1.2 for static state and 1.0 for seismic condition. The bearing capacity and the sliding in the foundation soil is dealt with by the manners mentioned in § 1. In addition to the above mentioned analyses it is desirable to take the following measures to prevent the damage at the corner and the approach of the quaywall and at the joint of different structures: (1) increasing design K-value by some 20% (2) providing stays or (3) connecting the walls in the direction of quaywall.

2) Sheetpile bulkheads

The aseismic design of a sheetpile bulkhead is performed on the following steps.

(1) Computation of lateral earthpressure on the bulkhead:
(cf.2) and 3) in §1)
(2) Estimation of necessary length of sheetpile embedment:
120 % of the length computed by the free-earth support method is adopted.
(3) Design of tie-rod:
Tie-rod tension is computed by supposing the bulkhead as a simple beam which is supported at the sea bottom and the position of wale. Allowable stress of tie-rod is to be 900 kg/cm² and 1400 kg/cm² for static and seismic conditions respectively.
(4) Design of sheet-section:
The maximum moment is computed for the simple beam mentioned above. Allowable stress of sheetpile is 1400 - 1600 kg/cm² and 2100-2400 kg/cm² for static and seismic conditions respectively.

These computations are illustrated in the design charts given in the manual in the case of a sheetpile bulkhead with relieving platform, horizontal forces on the bulkhead including horizontal mass force of the platform and the fill on it are supported by batter piles, design of which is the same as described in section 5) in § 1.

3) Cellular bulkheads

In the aseismic design of a cellular bulkhead, the stability analysis of bulkhead itself should be included in addition to the analyses of the whole structure as a gravity type quaywall. Since the stability of the

bulkhead is considered to be depending on the lock tension of sheetpiles and internal shear of bulkhead, analyses should be performed on these items. The width of bulkhead is so determined that the shearing force S on the vertical neutral plane of bulkhead does not exceed the allowable shearing resistance Sa of bulkhead. The values of S and Sa are given in following equations.

$$S = \frac{3}{2}\frac{M}{b}$$

$$Sa = \frac{1}{F} \times \frac{1}{2} Ki \cdot \gamma \cdot H^2 (\tan\phi + f)$$

where

M : Overturning moment due to external force acting on the bulkhead above the sea bottom
b : Width of bulkhead
F : Safety factor
Ki: Lateral earthpressure coefficient of fill material (usually 0.6 is used for both static and seismic conditions)
γ : Unit weight of fill material
ϕ : Angle of internal friction of fill material
H : Height of the wall from the sea bottom
f : Frictinal coefficient between the locks of sheetpiles

The lock tension T given in the following equation is compared with the allowable value, which is 250 ton/m for the straight sheetpile produced in this country.

$$T = Ki, \gamma, H\, r$$

where

r : Radius of a circular cell

4) Trestle type piers

Earthquake force to a trestle type pier is only mass force of the trestle itself and surcharge on it . The following assumptions are made on the fixity of the embedded part of the pier. (1) For the trestle type pier of piling, the depth of the point of fixity is assumed to be αH, in which H is the height of trestle (neutral axis of beam) from the sea bottom and α is coefficient being equal to 0.1 for sandy soil and to 0.25 for the sandy clay or firm clay ($c > 0.3 kg/cm^2$). (2) In design of trestles of cylinder supported by piles, pile heads are assumed to be hinged. (3) In the case of the pier which has trestles of large dimentions (caisson, pneumatic caisson or well) the design of trestle is similar to that of a well.

It is a common practice to rigidly connect the trestle with the floor frame, but it is desirable to divide a pier into several blocks to prevent the different settlement. In such a case a dynamical analysis can be applied to each block, because it is considered to be a relatively simple system of vibration.

5) Trestle type piers with small retaining walls

In addition to the separate analyses of the main part and the retaining wall, the stability of a whole structure including slope and wall should be analyzed according to the procedure shown in the section 6) in preceding

Chapter 6. Bridges

§1. Earthquake Damage

The following are the general findings of the survey of the damages which bridge structures suffered from five violent earthquakes occurred during the last four decades in Japan: the Kanto Earthquake (September 1, 1923; M = 7.9), Nankaido Earthquake (December 21, 1946; M = 8.1), Fukui Earthquake (June 28, 1948; M = 7.2), Imaichi Earthquake (December 26, 1949; M = 6.5), and Tokachi-oki Earthquake (March 4, 1952; M = 8.2).

a) No damage was found in regions of seismic intensity scale III (8-25 gal). Some little damages were found regions of seismic intensity scale IV (25-80 gals), and severe damages were caused in regions where seismic intensity scale V (80-250gals) or up. Structures located in regions of seismic intensity scale VI (250 - 400 gals) suffered much more extensive and severe damages than those in regions of seismic intensity scale V.

b) The degree of damages is largely related to the kind of foundation, and marked damages were observed where foundations are alluvial or soft ones.

c) As regards the aseismatic property of bridges, superstructures and substructures shall not be considered independently. For instance, while we can find some cases where the superstructure seems to have fallen down due to the damage of the substructure, there are cases where slanting or overturning of substructures shall be attributed to the extraordinary forces resulting from the downfall of the superstructure.

Details of earthquake damages of bridge structures are as follows.

1) Superstructures: Most of the damages are the downfall of girders or trusses. For example, there were some reinforced concrete girders that at last fell after moving perpendicularly to the bridge axis, and also some trusses that toppled sideways moving on rocker bearing or that fell to the ground because of the dislocation from piers. Worthy of mention is the Yoshino-River railway bridge that suffered in the Nankaido Earthquake. The superstructure of this bridge is a three-span (3× 71.2 m) continuous through truss of single track supported by piers of 20 m deep caisson foundation. In the case of this bridge, complex movement of pier tops due to the slanting and sinking of substructures caused relative displacements of truss shoes, both horizontally and vertically, which induced in the truss members stresses greater than the allowable stress. However, these stresses fortunately seemed to have not reached the yielding stress of the material and the bridge has been still in use after repairing.

There are also a few examples of reinforced concrete girders in which cracks grew due to the use of poor concrete having low strength.

2) Piers: The earthquake damage to piers is classified into the occurence of cracks, the cutting of pier body, the overturning after cut, the sinking and lean of pier, and so forth. In the bygone earthquakes, the damage occurred mainly in the piers made of bricks, the stone masonary piers

with filling, or the plain concrete piers, and on the other hand the piers made of reinforced concrete have hardly been injured by earthquakes.

Most of horizontal cracks in a pier body generated in the neighborhood of ground surface, but the number of cut sections tends to increase with the increase in height of a pier. The damage example of bridge structures in the ruins of the Kanto Earthquake of 1923 show that the number of cut sections was 2 or 3 if the pier height was 14 m - 17 m, while a pier of about 10 m high was cut at one section and the piers with less height were not cut horizontally by the earthquake shocks.

Even the similar piers at the same bridge site revealed different states of damage according to the nature of foundations they stood. The damage to such a pier as tied with arch on two wells was found not a little. Some of the piers constructed on a slope were considered to lean or overturn at their foundations because of the sliding of the slope, the flow of sand and so on.

3) Abutments: The damage to abutments from earthquakes are roughly divided into the lean of structures, the cutting of parapet wall, the horizontal cracks and cut of an abutment body, the damage of wing masonary, the bending or cutting of anchor bolts.

Judging from the examples of the damage from the bygone earthquakes, the stability of an abutment against seismic action is improved and the damage is decreased if the portion of the abutment buried in soil is large.

Girder acts as a strut against the sliding of an abutment, and consequently, there have been many damage examples of the bending and cutting of anchor bolts or the cutting of a parapet wall.

Moreover, the damage to foundations which include wells and piles, shall be discussed and it is suggested to refer the publications of the Japan Society of Civil Engineers regarding these items.

§2. Aseismic design method

As the design specifications for highway and railway bridges in Japan are based on the similar idea, discussions will be made mainly with railway bridges here.

The seismic force applied to a structure is assumed as a statical force which is obtained by multiplying the weight of the structure by seismic coefficient. The earthquake-resistant design of structures is specified in the draft of the Design Specifications for Plain and Reinforced Concrete Structures (April 1955) proposed for adoption by the Japanese National Railways. According to these standards, the horizontal seismic coefficient is taken as 0.15, 0.20, 0.30 dividing the country of Japan into three portions A,B, and C, and the vertical seismic coefficient value to be used is half the value of the horizontal seismic coefficient. These coefficients can be modified by the use of table A and B, if the designer so desires and only with the approval of the railway authorities, to account for such items as the locality of the structure, the type of site soils, the kind of structure,

a balance between strength and stability of structure, and the respective vibrational characteristics for the structure and the foundation.

TABLE-a
Horizontal Seismic Coefficient Modification Table

Type of Caluculation	Strength Calculation of a structure which vibrates freely						Stability calculation of a structure which vibrates freely, and strength and stability calculations when the effect of earth pressure is taken into consideration		
	Massive Structure			Slender Structure					
Region / Kind of Foundation	A	B	C	A	B	C	A	B	C
Class I	0.35	0.25	0.20	0.20	0.15	0.10	0.20	0.15	0.10
Class II	0.25	0.15	0.10	0.30	0.20	0.15	0.25	0.20	0.15
Class III	0.15	0.10	0.10	0.30	0.20	0.15	0.30	0.20	0.15
Class IV	0.15	0.10	0.10	0.30	0.20	0.15	0.35	0.25	0.20

TABLE-b
Classification of Foundation Soils

Class	Kind of Bed
Class I	The surface layer is an Alluvium formation 2 meters or less in thickness and directly underneath exists a hard stratum of the tertiary or earlier era extending over a fairly wide area.
Class II	The surface layer is of the Diluvium formation whose thickness ranges from 3 to 15 meters or of the Alluvium formation whose thickness ranges from 2 to 10 meters
Class III	The surface layer is of the Diluvium formation 15 meters or more in thickness or of the Alluvium formation, whose thickness ranges from 10 to 25 meters.
Class IV	Remarkably soft and weak bed, or the surface layer is of the Alluvium formation, 25 meters or more in thickness.

These coefficients are to be changed in consideration of the grade of materiality of the railway lines.

On the other hand, the Design Specifications for Steel Railway Bridges (Japanese National Railways; 1955) specify the standard seismic coefficient o 0.2 (horizontal) and 0.1 (vertical) for design of steel railway bridges. However, these values can also very in consideration of the conditions of

bridge site. The reasons why the different values of seismic coefficient are employed for concrete structures and steel railway bridges are as follows: first, the fatal damage to steel bridge superstructures from an earthquake has not been reported so far, and second, if the constant seismic coefficient is used, the identical standard design is usable throughout the country.

Inasmuch as there has been no generally reliable theory developed or adequate actual observed date with respect to the earth pressure during an earthquake, there is no way to pin-point the resultant earthquake effect other than to use the results of experimental tests and what actual observations have been made. At the present stage, Dr. Mononobe's theory on the earth pressure under seismic action is widely used. Although the assumptions made by Dr. Mononobe might be valid at some time, the earth pressure during an earthquake varies according to the complexity of the vibration and the deformation condition of each part. Accordingly, it shall be necessary to take such items as the rigidities of foundations and structure, the characters of soil in contact with them, to find out the earth pressure during an earthquake.

(NOTE) In Mononobe's theory, the foundations and structure are regarded as one body and are tilted in the critical direction, in respect to the direction of gravity force, an angular amount of $\theta = \tan^{-1} K$. Then, making use of $W' = mg\ (1 - K_v)\ \sec\theta$ instead of $W = mg$, the earth pressure is calculated by means of a method similar to that for the normal gravitational earth pressure. In the above expressions, $K = K_h/1 - K_v$ is the resultant seismic coefficient, where K_h = horizontal seismic coefficient and K_v = vertical seismic coefficient.

So far as the stability calculations for structures are concerned, the draft of the Design Specifications for Plain and Reinforced Concrete provide for the stabilities against sliding, overturning, and bearing capacity.

The safety factor for sliding at the bearing face and the base of foundation etc. is specified to be more than 1.5 and the resistance to sliding of pile foundation is taken as equal to the resistance at foundation base. When the battered piles are used, the resistance of soils may be added to the allowable horizontal resistance of the piles to sliding, which is defined by the horizontal component of the vector in the direction of pile axis expressing the allowable bearing power of the pile.

In regard to the safety for overturning, the point of application of the resultant force of loading on foundation base shall be in a middle third of the base width if a structure stands on soil, and be in a middle half of the base width if it stands on firm foundations like rocks or piles. When a foundation is soft and it has a fear of unequal settlement, the point of application of the resultant force mentioned above is desirable to be as close to the center of foundation base as possible. However, in designing a substructure against earthquakes, the point of application of the resultant values of foundation reaction or loading at the top of a pile exceed the allowable bearing capacity. As a rule, the normal allowable bearing capacity of soil foundations and piles is a half of the ultimate bearing capacity on the bases of the actually measured values, but the allowable bearing capacity of the clay foundation and the friction piles supported by unstable soils like clay shall be less than one -third of the ultimate value.

These allowable bearing capacity employed for aseismic design should be the value at the normal times.

The allowable stresses of materials at the time of an earthquake is differently specified for concrete structures and steel bridges. In the former, the allowable stresses may be increased 50 % for combinations of seismic and dead loads and increased 100 % for combinations of seismic, dead and live loads. On the other hand, in the design of steel bridges, the allowable stresses may be increased 75 %. This difference is originated from the fact that at the time of an earthquake the stress of steel is permitted up to its yielding stress in designing steel bridges.

The actual structures of prestressed concrete have a history of only about six years in Japan, so there is no example of earthquake damage to them and they are designed in the same way as for reinforced concrete structures. The relation between the natural frequency measured and span of the prestressed concrete highway bridges ever constructed is shown in the following table.

Span (in m)	15	20	25	30	35	40
Nat. Freq. (cps)	6.0	4.5	3.5	3.0	2.7	2.4

Since one can think of a fear that they might be in resonance with earthquake motion the amount of anchor bars is calculated by the same computation method as in reinforced concrete highway bridges and the girders are anchored in their substructures.

§3. The earthquake-resistant practice of bearing construction in bridges

The evidence obatined from past earthquake damages show that a superstructure of bridge was hardly injured by itself and the damage was found mainly in its substructure or the connection between super and substructure. To cite the instances of the latter, because of the lack in strength at the connection a girder jumps on to substructure or falls to the ground, and sometimes rollers at support are out of place.

In the light of these earthquake damages, the following countermeasures have been taken in earthquake-resistant construction of supports of railway bridges.

(1) Supports of truss bridge: In 1916, the oldest type of the bearing construction was adopted as a standard design of the shoe of truss bridges by the Japanese National Railways. However, based on the experiences from the Kanto Earthquake of 1923 the modification of design was made and then a new standard design appeared in 1925 since the modification mentioned above was not satisfactory. In the new standard design, a pin was covered by a cap to connect the upper shoe with the lower one and the shoe was anchored to the body of substructure by bolts in order to prevent truss from jumping.

The design to keep rollers from being out of place, has been used since 1937. In this practice, because the projecting parts on and underneath the chain lock is inserted into notches of both shoes with some margin, rolling-out of the whole rollers kept their spacing by a connection plate is prevented without having a hindrance to a move of the roller.

(2) Supports of plate-girder : The structure of a support of plate-girder bridge has been also modified from time to time. In 1919 and 1920, the Japanese National Railways adopted a standard design (E33 and E40 Deck plate-girder) in which there was no anchor bolt but in the light of the experiences from the Kanto Earthquake of 1923, the anchor bolts to connect the girder to a substructure were adopted after 1924.

(3) Steel post supporting plate-girder : In the overbridges constructed in 1907 through in Tokyo by the Japanese National Railways, the post of a plate-girder had two spherical bearings made from cast steel at its both ends. Since this structure had a defect in keeping the girder from jumping during an earthquake, pins were inserted at the both ends of a post to prevent uplift after 1924.

Later the spherical bearings were employed again, and the modification has been made at the point that pins were inserted at the top of spherical surface. Although there remains some ambiguity in stress distribution, this practice has been used so far in Japan.

Chapter 7. Conclusion

In the above chapters, general standards applied to the usual structures are described, However, the specially important structures are used to be designed after precise investigations on their stability during earthquakes. Some examples of the procedures for modern aseismic design are as follows.

1) The New East Pier in Port Kobe :
Seismic intensity to be expected at the site was determined by refering to many past earthquake records obtained by the Kobe Mateological Observatory at its location, considering the difference of geological conditions of both location. Coefficients of base shear for each floor were decided by theoretical investigation on the vibration of structure. After the completion, various tests on the vibration characteristics of the structure were carried out.
2) Wakato Suspension Bridge :
Wakato Bridge is under construction at present and is 90 m + 364 m + 90 m in span length and will become the longest bridge in Japan after completion. In order to examine its seismic stability, vibration tests on the model which is 1/100 of the prototype in scale was carried out. The movement of the foundation during earthquake is assumed to be represented by a sinusoidal wave, because there is no record of strong motion earthquake at Wakamatsu, and nevertheless we must design it dynamically, for fear that there occurs resonance of the bridge with the ground motion during an earthquake. The amplitude, period and the number of ground motion of sinusoidal wave are decided theoretically as well as experimentally.
3) The Kamishiiba Arch Dam :
The approximate period of vibration is given by vibration test using the models, and the seismic coefficient is determined by using the record of earthquakes at the dam site. After completion, vibration due to earthquake are measured, and the conditions for design are checked by the records obatined by the seismograph.
4) Bridges in the Meishin Express Highway:
Tremor observation was done at the site of bridges planned, and the proper seismic coefficients are determined on the basis of its results.

In order to design structures reasonably, it must be necessary to get the more detailed and complete informations on characteristics of the earthquake, and dynamical properties of materials and structures. And now field and laboratory tests, and theretical researches are being carried out for the purpose of getting the more attainments.

DESIGN SEISMIC FORCES FOR REINFORCED CONCRETE BUILDINGS

by YUKIO OTSUKI

INTRODUCTION

The Japanese Code requirements for the design seismic force specifies the use of a seismic coefficient, K, calculated as a percentage of the gravitational acceleration, g. The Code requirements were contained in an article by the writer presented to the First World Conference on Earthquake Engineering held at Berkley, California in 1956 (1).

This form of specification is simple to apply to actual design calculations and gives reasonable results for average building structures as long as the buildings are squatty and fairly rigid. But the writer feels somewhat sceptical about the validity of applying the same requirements to non-squatty or slender buildings, the vibrational behaviors of which are obviously so different.

The design seismic force for a particular building structure should be determined from the vibrational characteristics of that building. A remarkable achievement in utilizing such building characteristics is the recent San Francisco Building Code requirements in which the design base shear coefficient for a building structure is given as a function of the structure's natural period of vibration. The form of representing this force coefficient is as percentages of gravity acceleration similar to other existing Code requirements.

Up-to-this time, therefore, whether or not the natural period of vibration of the structure is taken into account, all Code requirements specify the statical equivalent coefficient which is expected to give appropriate values for the design storey shears. Sometimes this procedure results in excessive overturning moments for the lower portions of the building, particularly as the height of the structure increases. Some special procedures become necessary to handle such large moments.

A fact that earthquake ground motion is not steady indicates that a building structure will never vibrate accordingly during an earthquake. Therefore, the most adverse conditions will never occur simultaneously throughout the building structure. If the coefficients are so specified as to insure sufficient strength for the upper portions of a building, then the lower portions of the same building certainly will be overdesigned, and vice versa. Such contradicting tendencies can be eliminated by the introduction of some localized design conditions in addition to overall requirements.

Further, current design methods in which a building structure is assumed to deform in the shearing mode during an earthquake have never been proved inadequate. Supposing therefore, that a certain storey of a building structure is deformed in the shearing mode, it is obvious that the force

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induced in this storey will be equal to the product of the amount of storey deformation and the storey rigidity.

Therefore, when the vibrational pattern of a building structure can be determined with reasonable accuracy, it is possible to calculate all storey deformations, and hence the storey shears if all storey rigidities are known.

This becomes the basic philosophy of this proposal. The important quantities in this proposal are:

Natural period of the structure
Natural period of the site ground, and
Design velocity of the structural vibration.

GROUND MOTION AND STRUCTURAL RESPONSE

In order to cope with the problem dynamically, it is necessary to define the ground motion during an earthquake. As described previously, the ground motion during an earthquake is not stationary. The best way to define the effect of the ground motion is to define the structural response instead of the ground motion itself. Then, considering the ground, or subsoil, to be an elastic medium, a stratum or strata has its own natural period of vibration. Accordingly, a structure which is built on this soil will be excited in its natural modes of vibration by the earthquake ground motion which has been modulated by the vibrational nature of the site strata.

This is equivalent to what Messers. Martel, Housner and Alford maintained in the discussion of a relatively small slender tower on top of a building (2). Mr. H. Tajimi has made an extensive study on this problem (3). He has concluded that tendencies coinciding with Housner's results (4) occur only if the abscissa of Fig. 6 of Reference (4) is changed into the period ratio of the structure to the ground, Ts/Tg, in place of the simple structural period, Ts.

The response velocity certainly will vary with the structural damping. However, for practising engineers, it is a difficult problem to assume an appropriate value of damping for any structure under immediate consideration. However, if the scope of application is limited to cover only reinforced concrete structures, with or without encased fabricated structural steel frames, the range of damping will be limited to within a comparatively narrow range that can be deducted by formula.

It is well known that shear walls placed in a building structure are very effective in minimizing the structural deformation under earthquake loading. The shear walls will be the first elements to become overstressed when the building vibrations become violent. They then dissipate energy and prevent the rest of the structure from becoming greatly overstressed. Further, it has been observed that when shear walls have been provided in sufficient numbers in a building structure, the amount of shear deformation is reduced to a remarkable extent as compared to the integral-unit movements such as rocking and swaying displacements. Both of these shear wall factors mean an appreciable increase of damping.

Soil conditions must be carefully considered also; past experience shows that the intensity of the earthquake shock is always greater on soft aluvium than on harder soils.

Mr. H. Kobayashi's studies indicate that the taller the structure, the greater will be the velocity of vibration. The empirical relationship between the height of the structure and the vibrational velocity can be expressed approximately as the two thirds power of the height.

Considering all the contributing factors expressed herein, it is proposed that the design vibrational velocity be calculated by use of Eq. (1).

NATURAL PERIOD OF A BUILDING STRUCTURE

In order to define the vibrational velocity of a building structure, the correct assumption of the natural period of the structure is of paramount importance. The requirements of the San Francisco Code gives an underestimation of the period giving a margin of safety. In this proposal, underestimation of the period will not result in such a margin of safety. Therefore, a more accurate formula for the period estimate is a necessity.

Accumulated vibration test data (6) clearly indicate that the majority of the existing buildings tested rock and sway, and behave differently from a fixed base condition. In the case of a shear beam, the natural period of the beam is theoretically proportional to the product of the square root of the tip deflection under the force of horizontal gravity and the length of the beam. It is to be emphasized that the deflection of the beam tip consists of two components: the deformation of the beam proper and the displacement due to the deformation of the support. Therefore, it is a suggestion of this proposal that the total displacement of the building top be divided into two parts: the elastic deformation displacement of the superstructure and the displacement due to the deformation of the substructure.

Another factor to consider is the fact that a very flat structure such as a machine foundation where the height, H, is practically zero, still has its own natural period, which is finite.

Considering all the pertinent factors, a formula something like Eq.(2) is developed.

In the determination of the magnitude of factors which represent structural deformation and subsoil deformation, it is best to avoid becoming too theoretical such as in the assumption of the rigidities of structural members or the spring constant of the soil. Eq. (2) is absolutely empirical, therefore, but never the less it can be expected to predict the natural period of a building within about twenty percent; which considering the possibilities of error is quite close.

The natural period so predicted is the natural period of small amplitude, or the initial period. It has often been noted that when the amplitude becomes large as in a violent earthquake, the period of a structure

will be elongated appreciably. This is probably due to localized damage of both superstructure and substructure. However, until such local damage does occur in a structural system, the structure will certainly be subjected to a force system consistent with the initial state.

In case where the period becomes elongated because of such local damages, such elongation will be inversely proportional to the square root of the rigidity. As long as the period ratio of the structure to the ground remains less than unity, the design velocity is proportional to the natural period, again inversely proportional to the square root of the rigidity. This means, the overall deflection, hence the storey deformation is inversely proportional to the rigidity. Now, since the storey shear is given as product of the rigidity and the deformation, the storey shear will remain unchanged as long as the natural period of the ground remains constant. For the range where the structural period is longer than the ground period, the design velocity remains constant, and therefore the elongation of the structural period reduces the design shear.

For reasons stated previously, it is proposed to use the initial period of a structure as the basis of the design analysis.

It may be pointed out that the natural period of the ground also will become longer in a violent earthquake. However, the elongation of the ground period always tends to the favor of the situation.

In this proposal, it becomes necessary to estimate the period of the second mode vibration. It must be born in mind that although the end conditions have appreciable influence on the fundamental period, they have but little influence on the second mode. Therefore, the second mode period can be computed using Eq. (2.6).

NATURAL PERIOD OF THE GROUND

Since the design velocity is given as a function of the period ratio of the structure to the ground, it is equally important to know about the predominant period of the ground. However, as mentioned previously, the accuracy of the ground period will not matter as seriously as in the case of the structural period. The elongation of the ground period tends to decrease the design shear, and therefore, it is proposed to take the initial period as the basis of the design analysis. Such predominant period of the ground can be determined from the frequency distribution of small ground tremors (7). Where such data is unavailable, provisions are made so that one can choose an appropriate standard value according to the formation of the subsoil strata. It is needless to say that the more accurately the ground period is estimated, the more reasonable the design shear that can be obtained. In any cases, the ground period should not be assumed as excessively long.

DEFORMATION PATTERN OF A BUILDING STRUCTURE

Since the natural periods of a building structure and the subsoil are known, it is possible to define the design velocity by assuming the quantity

of wall elements to be provided in the building refering to the structural plans. The next step then is to find out what deflection pattern of the building structure is both in the fundamental and the second modes.

This can be done only if the rigidity distribution along the height becomes known. This process may be quickly done by use of a computor. Even if such facilities are not available, the modified Stodola method (8) and Holzer method (9) will give both a quick and good approximation for design purposes for the fundamental and the second modes, respectively.

The next step then is to calculate each storey deformation. For the second mode, the storey deformation can be obtained readily from the deflection curve by subtracting the deflection of the certain storey in question from that of the upper and adjacent floor level. But in the case of the fundamental mode, since it contains also the bodily displacement due to the soil deformation, it is nacessary to calculate the pure structural deformation by subtracting that amount from the apparent deformation. When all of these procedures are complete, a table can be set up in which all of these storey deformations are listed.

DESIGN SHEARING FORCE

When all storey deformations are calculated, the last step is then to calculate the design shear for each storey. It is done simply by multiplying each storey rigidity by the corresponding storey deformation.

Now, there are two components for each storey deformation, one for the fundamental mode and the other for the second mode. The former is to be used to calculate a force system with the equilibrium of the complete building structure while the last is for local adjustment disregarding the equilibrium as a whole. This adjustment is applied in such a manner that the storey shear calculated from the fundamental mode is multiplied by the ratio of the sum of the absolute values of the fundamental and second mode storey deformations to the fundamental storey deformation. The adjustment is to be applied storey by storey, and the adjusted shear should not be considered as exerted simultaneously for all storeys.

A question may arise then as to what the appropriate value is for the rigidities of storeys to be used in this step. The answer is the rigidity which is consistent with the design natural period which is given by Eq. (2) of this paper.

In the course of calculating the vibrational mode, it is necessary to assume storey rigidities, or at least rigidity ratios for all storeys. But since this assumption is made quite independent of Eq. (2), the computed value based on those rigidities will not coinside with the predicted period. Therefore, correction as shown by Eq. (6) should be applied.

PROPOSED METHOD FOR DETERMINATION OF EARTHQUAKE FORCES

Scope of Application: The following provisions are applicable to framed reinforced concrete building structures with or without encased fabricated

structural steel which are taller than 16 meters above the ground; or to such a building the structural nature of which is comparatively different from the conventional structure and considered inappropriate for application of the current Code provisions.

Design Velocity: The design velocity is to be determined from the following formula:

$$V = C_1 \times C_2 \times C_3 \times V_0 \times \left(\frac{H}{30}\right)^{\frac{2}{3}}, \qquad (1)$$

where C_1 is a coefficient due to locality of the site, and same as specified in the present Code (5).

C_2 is a coefficient which is determined as follows;

Let T_S be the natural period of a building and T_g be the natural period of the ground, both in seconds,

a) For $\frac{T_S}{T_g} \leqq 1.0$, $\quad C_2 = \frac{T_S}{T_g}$, (1.1)

b) For $\frac{T_S}{T_g} > 1.0$, $\quad C_2 = 1.0$. (1.2)

c) For soft ground, the natural period of which is longer than 0.6 seconds, or if it is difficult to define its natural period, then

for $T_S \geqq 0.5$ seconds, $C_2 = 1.0$, and

for $T_S < 0.5$ seconds, $C_2 = 2\,T_S$. (1.3)

d) When the soil is definitely of hard formation, but is difficult to define its natural period, Tg can be assumed equal to 0.25 seconds.

C_3 is a coefficient which is a function of the wall quantity and is given as follows:

$$C_3 = 1.3 - \frac{w}{25}, \qquad (1.4)$$

where w is the wall quantity per unit floor area in terms of cm / m^2, in respective directions. The value for w shall be the minimum for all storeys in the respective directions.

V_0 is the basic velocity which is given as follows; according to the nature of the subsoil formation:

a) For tertiary or older 12 cm/s

b) For deluvial formation or alluvial formation, the thickness of which is not deeper than 5 meters 15 cm/s

c) For alluvial formation the thickness of which is equal to, or deeper than 5 meters 20 cm/s

Natural Period of Building Structure: The natural period of a building structure shall be computed by the following formula:

$$T_{S1} = 0.007 \ (H + 15) \sqrt{f_1 + f_2} \qquad (2)$$

where

$$H = H_0 + \frac{1}{3} H_u \quad (m) \qquad (2.1)$$

H_0 = Building height above the ground in meters.
H_u = Depth of basement in meters.

$$f_1 = 0.07 \ (20 - w) \qquad (2.2)$$

$$f_2 = K_1 \times K_2 \times \frac{1 + \lambda^2}{\lambda} , \qquad (2.3)$$

$$K_1 = (15 + w) H / 2000 \qquad (2.4)$$

for direct foundation on base rock, and

$$K_1 = (25 + w) H / 2000 \qquad (2.5)$$

for foundations other than above.

K_2 = Coefficient due to type of substructures given as follows:

Direct foundation on base rock 1.0

Bearing piles or piers

	Length in meters			
	0 - 5	5 - 10	10 - 15	15 and up
Piers	1.0	1.1	1.2	1.5
Piles	1.5	1.5	1.8	2.5

Friction piles 2.5

Direct foundation on other than rock 3.0 - F/25,
where F = Allowable bearing power in t/m^2.

λ = H/D, where D is the depth of building measured along the direction under consideration, in meters. Where H/D is less than unity, λ is assumed to be equal to unity.

$$T_{S2} = 0.4 \ T'_{S1} . \qquad (2.6)$$

where $T'_{S1} = 0.007 \ (H + 15) \sqrt{f_1}$. (2.7)

Observed periods for various buildings are shown in Table 2 in comparizon with the computed periods by Eq. (2).

Natural Period of Ground: The natural period of the site ground is to be determined by the frequency distribution data obtained from the microtremor observations at the site. Where such measurement is not feasible, the following figures can be used;

Hard rock	0.15 seconds
Soft rock	0.25 seconds
Deluvial formation	0.40 seconds
Alluvial formation	0.60 seconds

Determination of Design Shear:

a. Design velocities V_1 and V_2 and natural periods T_{s1} and T_{s2} are to be calculated for the fundamental and second modes of vibration by Eqs. (1) and (2).

b. Deflection patterns for the above mentioned modes, X_1 and X_2, are to be determined. The following constants then can be determined.

$$\beta_1 = \frac{\sum W_i X_{1i}}{\sum W_i X_{1i}^2}, \tag{3.1}$$

$$\beta_2 = 1 - \beta_1 . \tag{3.2}$$

c. Top deflections for the respective modes are determined as follows:

$$y_{1T} = \frac{V_1 T_{s1}}{2\pi} \cdot \beta_1 \cdot \frac{f_1}{f_1 + f_2}, \tag{4.1}$$

$$y_{2T} = \frac{V_2 T_{s2}}{2\pi} \cdot \beta_2 . \tag{4.2}$$

The complete deflections are then calculated for the respective modes as follows:

$$y_1 = y_{1T} X_1, \quad \text{and} \tag{5.1}$$

$$y_2 = y_{2T} X_2 \tag{5.2}$$

d. All storey deformations are to be calculated for respective modes.

e. Storey shears are to be determined by multiplying each storey rigidity by the storey deformation obtained. The rigidity figures as herein used must be consistent with the natural period used in (c).

Let R be an assumed rigidity and R' be the corrected rigidity which is consistent with the natural period used in (c). Denoting the computed period basing on R by T_m, and the period as given by Eq. (2.7) by T'_{s1}, then it follows:

$$R' = \left(\frac{T_m}{T'_{s1}}\right)^2 \cdot R \tag{6}$$

f. The fundamental mode shear is to be used for calculation of the equilibrium state of the building as a whole, while the local shear, which is the sum of the absolute values of the fundamental and second mode shears, is to be applied locally to each corresponding storey.

NUMERICAL EXAMPLE AND COMPARISONS

A sample calculation in accordance with this proposal has been carried out and the results are compared with current Code values, and additionally with triangular distribution in which the base shear coefficient is adjusted to be equal to 0.2.

It is observed, as shown in Fig. 1, that this proposal requires more

shear for the upper part of the building, but less for the lower part than has been required by current Code. The general tendency of the design storey shear distribution is similar to one calculated from triangular distribution. However, a comparative difference can be seen between these two for the overturning moment distribution. Such tendency is expected to become remarkable as the building height increases. The shear distribution pattern is generally similar to the results in References (10) and (11).

Because of the factor C_2 , as the building height becomes taller and the natural period of the fundamental mode becomes longer than the period of the ground, the ratio C_{i2}/C_{i1} will increase, i.e., the contribution of the second mode is expected to become pronounced.

CONCLUDING REMARKS

This proposal was drafted initially by the author and has been discussed at the meetings of the Vibrational Committee of the Architectural Institute of Japan. Messers. K. Kanai, T. Hisada, K.Nakagawa, H. Kobayashi, N. Ando, T. Tajime, H. Tajimi, Y. Osawa, Y. Murata, T. Iwashita and T. Fukuchi participated in the discussions. To date, the committee has not been able to reach a final conclusion. However, the author believes that the following three points have been generally accepted by the committee members:

1) The design shear for a building structure is to be determined according to its vibrational behaviors, such as the natural periods of both the building and the ground.

2) The introduction of local treatment presents a good solution for the present problem.

3) Eq. (2) for the prediction of the natural period of a building gives a good approximation.

One of the main points of those discussed at the meetings was that the proposal seemed to be a little too complicated to be uniformly applied as a Code requirement. Another point was that the requirements in the proposal were not uniform; some parts are too theoretical as compared with some other more practical parts. Some members prefer to give a seismic coefficient form to the present deformation and rigidity proposal.

These discussions are very worthy in that they represent the thoughts of men fully qualified. The scope of application is therefore limited to tall or otherwise unconventional building which are best designed only by well qualified structural engineers.

The non-uniformity of the requirements can not be avoided because of the nature of the proposal, unless of course all emperical requirements can be replaced by those more theoretical and competently handled. It is to be stressed that this proposal tries to specify requirements in simplest form as long as is possible without such simplification resulting in the invalidation of the problem.

It would not be impossible to specify the design seismic factor in

terms of seismic coefficients, with a certain adjustment for local shear due to the higher modes, if a simple formula can be established to give appropriate values. The author feels, however, that a representation such as seismic coefficients would not be effective for any building structure in which the rigidity distribution along the height is unconventional.

This proposal requires more time for design shear calculations than engineers are required to expend when following the simpler method delineated by most present Codes. The time required for the calculations following the method outlined herein will be somewhere between two to six hours according to experience. This is, of course, only a fraction of the time required for the complete structural analysis. It is believed that the added time spent compensates in that it gives the engineer a more complete grasp of the general structural behavior pattern of the building during an earthquake.

The author hopes that this proposal will help to provide a design method satisfactory enough to alleviate the present Japanese Code restrictions against buildings for the taller buildings of this type of construction, although the Code restriction is not for the structural reasons. It is hoped also that it may help those persons in other seismic countries who are forced with the same problem.

ACKNOWLEDGEMENT

The Author wishes to express his deep appreciation to the members of the Vibration Committee of the Architectural Institute of Japan mentioned in this paper for their cooperation and advice and especially for their generosity in allowing the use of their finding in this paper.

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NOMENCLATURE

β_1	Coefficient of excitation, fundamental mode	
β_2	Coefficient of excitation, second mode	
C_1	Coefficient due to locality of the site	
C_2	Coefficient due to period ratio of building to the ground	
C_3	Coefficient due to wall quantity	
D	Depth of building	m
F	Allowable bearing capacity	t/m^2
f_1	Factor representing structural deformation	
f_2	Factor representing building displacement due to foundation yield	
H	Building height	m
H_o	Building height above ground	m
H_u	Depth of basement below ground level	m
K_1	Coefficient representing foundation yield	
K_2	Coefficient representing foundation yield	
λ	Inertia factor	
M	Overturning moment	t-m
Q	Design shear	t
R	Rigidity of building	t/cm
R'	Corrected rigidity of building	t/cm

T_g	Natural period of the ground	sec
T_S	Natural period of building	sec
T_m	Period corresponding to assumed rigidity	sec
T_{S1}	First mode period	sec
T^*_{S1}	First mode period for fixed base	sec
T_{S2}	Second mode period	sec
V	Design velocity	cm/s
V_o	Basic velocity	cm/s
V_1	Design velocity for fundamental mode	cm/s
V_2	Design velocity for second mode	cm/s
w	Wall quantity in one direction	cm/m^2
W_i	Load at i-th floor level	t
X_1	Normalized deflection, fundamental mode	
X_{1i}	Normalized deflection at i-th floor level, fundamental mode	
X_2	Normalized deflection, second mode	
X_{2i}	Normalized deflection at i-th floor level, second mode	
y_1	Design deflection, fundamental mode	cm
y_{1T}	Design deflection at the top of building, fundamental mode	cm
y_2	Design deflection, second mode	cm
y_{2T}	Design deflection at the top of building, second mode	cm

APPENDIX

The welknown Stodola method was primarily prepared for the bending vibration of a beam, but the same principle can apply to the vibration of a shear beam.

The following tabular form is self explanetory and convenient for the mode calculation for the fundamental mode.

(1) N	(2) y_n	(3) W_i	(4) (2)x(3)	(5) $\downarrow\sum$(4)	(6) D_i/h_i^2	(7) (5)/(6)	(8) $\uparrow\sum$(7)	(9) (2)/(8)
R	-----	-----	-----				-----	-----
				-----	-----	-----		
N	-----	-----	-----				-----	-----
				-----	-----	-----		
N-1	-----	-----	-----				-----	-----
				-----	-----	-----		
---	---	---	---	↓			---	---
							↑	
---	---	---	---				---	---
2	-----	-----	-----				-----	-----
				-----	-----	-----		
1	0	-----	0				0	

In the above Table, column (8) gives the figure for y_{n+1}, and D_i denotes the rigidity figure as given by $D_i = R_i/(12EK_0)$.

The similar calculation is to be repeated until all of $(\frac{y_n}{y_{n+1}})_i$ values become uniform. Then, it follows

$$\left(\frac{2\pi}{T}\right)^2 = \left(\frac{y_n}{y_{n+1}}\right) \times 12EK_0 g.$$

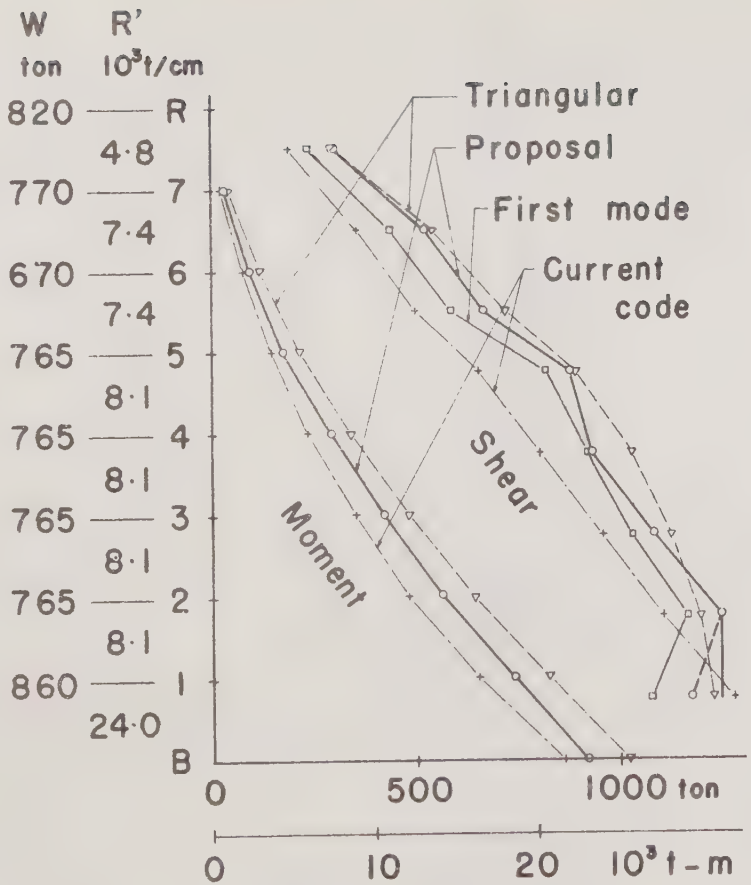

FIG. 1 Design Shear and Over-turning Moment.

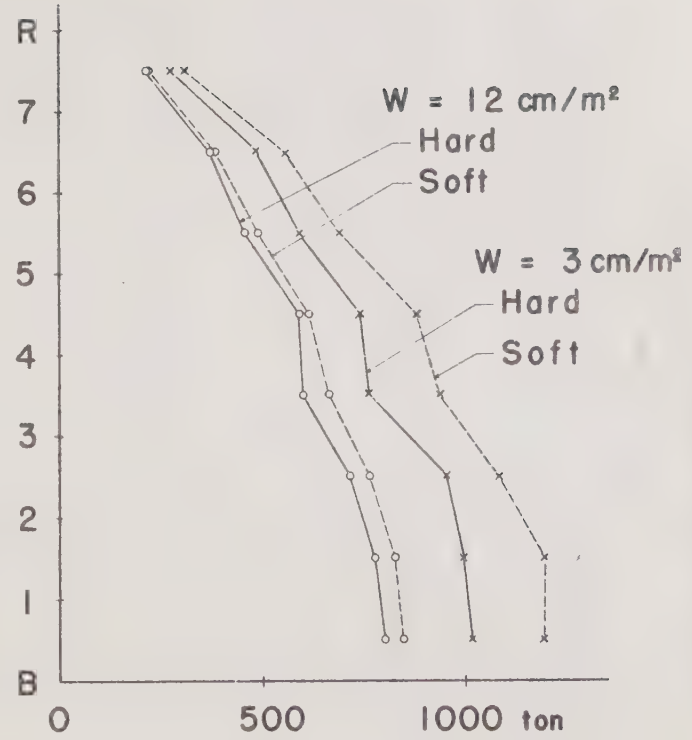

FIG. 2 Effect of soil and wall quantity on design Shear.

Table 1. Comparison of Eq. (2) with Observed Periods.

No.	Direction	H_o (m)	D (m)	N	H_u (m)	w (cm/m)	Foundation	Eq.(2) (s)	Observed (s)	Measured by
1	NS	31	35	9	17.35	7	Direct, tertiary	.478	.43	Naito
	EW		53			10		.447	.39	
2	NS	29.2	7.12	9	5.64	5	Pier, tertiary	.58	.575	Naito
3	EW	29.1	23.1	8	5.7	5	Bearing pile, 17m	.60	.63	Naito
4	Trans.	20.7	13.1	7	5.5	7	Direct, tertiary	.31	.302	Naito
	Long.		12.35			10		.306	.27	
5	Trans.	9.04	5.5	3	1.4	16.7	Bearing pile, 6m	.16	.18	Kanai
6	Trans.	9.04	5.5	3	1.4	16.7	Direct, hard rock	.134	.108	Kanai
7	EW	24.8	26.3	7	4.08	6.9	Bearing pile, 12m	.45	.38	Taniguchi
	NS		26.1			3.9		.455	.42	
8	Trans.	11.1	6.76	4	1.8	20	F = 8 t/sq.m.	.23	.32	Kanai
9	Trans.	11.1	6.7o	4	1.8	20	Friction pile	.23	.28	Kanai
10	Trans.	20.7	14.4	6	1.6	7.5	Friction pile	.40	.43	Tajimi

TABLE 2

	COMPRESSION	TENSION
STRUCTURAL STEEL	2400 kg/cm² (34200 psi)	2400 kg/cm²
CONCRETE	2/3 OF COMPRESSIVE STRENGTH	1/10 OF ALLOWABLE COMP. STRESS

DYNAMICAL COMPRESSIVE DEFORMATION AND FAILURE OF CONCRETE UNDER EARTHQUAKE LOAD

T. Hatano*
H. Tsutsumi**

Introduction

Study on the earthquake proof of concrete dams has expanded its scope from dynamics of perfect rigid bodies to the analysis of elastic bodies and even to elastic vibrational approaches in which the damping of vibration, to be obtained through vibration tests of real dams, is supposed to be viscous damping[1].

To advance such studies further, it seems highly important to make basic researches on dynamical properties of materials in the structure.

As the first step in this direction, we conducted experiments on the response of concrete when one stroke of compressive load, with the same load-velocity as earthquake load, is given, and attempted to look into the elasticity, viscosity and failure of concrete.

Such experiments were first attempted about 20 years ago by Professor Katsuta of Tokyo Industrial College[2], but we carried out manifold experiments on various kinds of concrete and mortar with a simple high-speed compressive test machine, which we made, and handy strain and pressure meters, and tried to obtain general relations, and furthermore to explain the dynamical behaviors of concrete.

Methods of Experiments

Photo 1 shows arrangement of the high-speed compressive test machine and other measuring apparatuses. It is possible to carry out experiments on failure at varying speeds ranging from 15 r/min. to 300 r/min., which will be developed by turning the fly wheel with motor. The oil pressure pump is driven by the fly wheel. The oil in the cylindrical oil tank flows into the pump, and by pulling the lever, the globular valve comes off and the compressed oil runs beneath the ram of the 100-ton Amsler compressive test machine. The ram is impulsively pushed up by the oil, and the specimen is compressed. On the other hand, the cum, linked with the piston of the oil pressure pump, is set in motion, and pushes the globular valve back to its original position so that the oil can be sent beneath the ram only during one stroke of the pump. The ram is hoisted by 2 mm in one stroke, and the 20 cm high concrete specimen fails in about 0.03 sec. to 1 sec. according to the speeds of the fly wheel ranging from 15 r/min. to 300 r/min.

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Photo 2 shows the positions of the strain and pressure meters for the specimen. As pressure meter, we used a short cylinder made of special steel, in which un-bonded type strain gauge was fixed. Its capacity is 100 tons, and sensitivity, 28 μ/ton. As the strain meter, we used π gauge, that is a thin phosphor-bronze plate, of the Ω shape on which a bonded type strain gauge is stuck. And the strain meters were fixed in the perpendicular direction on the pressure plates on the upper and lower sides of the specimen, and the deformations were recorded, together with pressure, in 20 cm between the bottom and the top of the specimen.

As it is difficult to expect parallel movements of the upper and lower sides of pressure plates, we used a total of four gauges in two diametrical directions and took the average of four strain values. The gauges could stand 3 mm of compressive deformation, and their degrees of sensitivity, though different to some degrees, were approximately 1,600 μ/mm. These measuring meters were carefully checked before and after each experiment (the pressure meter was examined by the Amsler test machine, and the strain meter, by a 1/1,000 mm dial gauge).

The properties of cement and aggregates used are given in Tables 1 and 2. Using these data, we carried out a total of 14 experiments on the Test No.1 - No.10 concrete and Test No.11 - No.14 mortar of the mix proportions and ages indicated in Table 3. The properties of not hardened concrete and mortar right after the making of the specimen are also appended in this thesis. In the meantime the specimen is a cylinder, 10 cm in diameter and 20 cm in height, which had been cured in the water at a temperature of 20 degrees C until right before each experiment.

Test Results

Each test was made on about 20 specimens, the first three of which were for ordinary statical tests. Although we designed to fail the specimens in about 100 sec., it was found difficult to adjust load velocity by hand. In dynamical tests, there arose cases where in two strokes of impulsive load acted on and failed the specimens or the records of the strain meter vibrated too much to read rightly. In the former cases, the test findings were discarded, and in the latter cases, only the readings of the pressure meter were adopted.

Failure, as is shown in Photo 3, typically gave conic dents on both top and bottom sides of the cylindrical specimen, but made no difference by failure time. But in the statical failure of a poor-mix specimen, just as shown in Photo 3, the specimen did not break up even after the failure. In the case of dynamical failure, it was found that the higher the age of the material was and the richer its mix was, the bigger sound was produced and the circumference of the conical dents was scattered in small pieces. It was also discovered that the larger the strength was, the more coarse aggregates were broken into pieces and the greater the deviation of strength was (See below).

Fig. 1 shows a few instances of test records. Since the load apparatuses made use of the rotation of the fly wheel, the relationship between load and time approximated to the (1 - cos at) form and near failure, the load velocity became low and the strain velocity high. In the case of

statical failure, any type of specimen and in the case of dynamical failure, the specimen of poor mix and long failure time, after withstanding a load slightly smaller than the maximum one after load reached the maximum point, were seen broken up. This tendency was far more conspicuous in mortar than in concrete. At the instant of failure, the pressure reading bounced back to its original position, while the strain record was seen vibrating at about 60 cycles/sec. due to the sharp rise of the ram.

The stress-strain curves were drawn up on the basis of test records, and the representative of them were described in Fig. 2-1 to Fig. 2-14 for each Test No. As the strain velocity is not constant, the curve are given by failure time t which is the time from the beginning of load to the maximum of it. The stress-strain curve of the portion which withstood load even after it reached the maximum stress and whose pressure record was clearly visible is indicated by dotted line. The diagram proves that the smaller the failure time is, the steeper the stress-strain curve crawls, and that the poorer the mix is, the more convex the curve becomes. The gibbous curvature of mortar is far larger than that of concrete.

Fig. 3-1 to Fig. 3-14 show the relationships between the strength of failure σ_u and failure time t and between the strain at the maximum strength of failure ε_c (which is defined as "compressibility") and failure time t. For failure times here are natural logarithmic values used.

It could be observed that almost linear relations exist between σ_u and lnt, and ε_c is almost stable regardless of t. Table 4 indicates regression line values and standard deviations in case σ_u and ε_c were adjusted in the light of these relations. The coefficient of variation of σ_u , if it is taken as the base when failure time is equal to 1.0 sec., is 4.7 to 3.0 % for concrete as against 2.1 to 1.8 % for mortar, and the tendency is seen that the poorer the mix, the smaller the coefficient.

The coefficient of variation of ε_c is 6.8 to 4.0 % for concrete as compared with 3.9 to 3.3 % for mortar. Secant modulus Es at points of 0.25, 0.50, 0.75 and 1.00 times ε_c on the stress-strain curve are given in connection with in Fig. 4-1 to Fig. 4-14. As the decline of Es is likely to slow down along with the increase of t, we assumed that Es is equal to $be^{-c\,\mathrm{lnt}}$, and the regression line values and standard deviations which were obtained on the basis of this relationship are given in Table 5-1 to Table 5-2.

The standard deviation of Es, though slightly larger than those of σ_u and ε_c, (t = 1.0 sec. is taken as the base) is 7.4 to 2.3 % for concrete 4.9 to 2.4 % for mortar. It is known here that the deviation of Es to a big strain value is rather smaller than the deviation of Es to a small strain value.

Consideration

When a concrete structure gives rise to compressive stress in its inside as a result of earthquake, its strain velocity or load velocity becomes unsteady and shows trigonometric functional changes. That is why we first deal herein with cases in which one stroke of trigonometric functional load velocity takes place. Hence it seems advisable to simply indicate

strength and the coefficient of elasticity in relation to the period of earthquake motion. In this case, it will be possible to, on the simple assumption that failure time equals half of the period of earthquake motion, apply the foregoing various dynamical tests at 0.03 to 1.00 sec. to conditions when the period of earthquake motion is 0.06 to 2.00 seconds.

From Fig. 3.and Table 4 can it be said that σ_u is in inverse proportion to t, and that the following equation can be established - with the same degree of accuracy as the compressive strength of concrete and mortar in statical tests:

$$\sigma_u = -d + f \ln t \qquad (1)$$

$$0.03 \text{ sec.} < t < 100 \text{ sec.}$$

In this test, supposing that the strength of 100 second failure is 1, it augments to 1.17 to 1.30 at the time of 1 second failure; and 1.30 to 1.56 at the time of 0.03 second failure. It is witnessed that the poorer the mix, the higher the multiplication rate tends to be, and that the multiplication rate is somewhat larger for the 13-weeks material than that for the 4-weeks material.

Compressibility ε_c becomes almost constant by the kind and age of concrete and mortar and the following equation seems to be established regardless of failure time:

$$\varepsilon_c = \text{Constant} \qquad (2)$$

$$0.03 \text{ sec.} < t < 100 \text{ sec.}$$

In this test, the compressibility, is 19.1 to 28.1 x 10^{-4} for concrete and 26.2 to 38.2 x 10^{-4} for mortar. These values are inclined to become larger when the mix is richer, and to be in proportion to the age of material.

These results of tests on compressive strength and compressibility seem to give valuable suggestions to the theory on the failure of concrete and mortar. When the theory on failure is considered in the light of time elements, the energy theory, much less the stress theory, seems to lose the ground. In this respect, the authors intend to introduce their views and tests on some other occasion.

How the incline of the stress-strain curve increases along with the decrease of failure time, as mentioned previously, will be able to be signified by the following equation using Secant-Modulus Es:

$$E_s = b\, e^{-c \ln t} \qquad (3)$$

$$0.03 \text{ sec.} < t < 100 \text{ sec.}$$

If Es at 100 seconds of failure time should be represented by 1, in this test it increases to 1.15 to 1.25 at 1 second of failure time and further to 1.20 to 1.50 at 0.03 second. It was found that the poorer the mix, the greater multiplication rate, and that the multiplication rate tends to be high towards Es against ε_c from Es against 0.25 ε_c.

With these data, it is possible to push rheological analysis of concrete as material. But as it will take a considerable amount of time to make a model which will suffice to satisfactorily explain the responses of concrete and mortar in various tests, we will limit ourselves to citing a few examples of constant values for a simple model.

As the consideration of all behaviors up to failure will necessitate the introduction of non-linearity, we will confine ourselves to strain and stress up to $0.25\,\varepsilon_c$ on the stress-strain curve. As a model, we use a simple three-element model, that is, the Kelvin model to which a spring is connected directly.

Now, if $$\sigma = K t \qquad (4)$$

should be acted on the model, strain ε will result, and the following equation will be established:

$$\varepsilon = K\left(\frac{t}{E_1} - \frac{\eta_1}{E_1^2} + \frac{\eta_1}{E_1^2} e^{-\frac{E_1}{\eta_1} t}\right) + K\frac{t}{E} \qquad (5)$$

in which

σ : Stress (kg/cm²) K : Constant (kg/cm²: 1/sec.)

E_1 : Coefficient of elasticity of Kelvin body (kg/cm²)

η_1 : Coefficient of viscosity of Kelvin body (kg/cm²: sec.)

E : Coefficient of elasticity of spring directly connected to Kelvin body (kg/cm²)

From the regression line of ε_c, σ_u, $E_{s,0.25\varepsilon_c}$ in the above diagrams will values at 0.03 sec., 0.2 sec., 1 sec. and 100 sec. be obtained; K will be obtained from the strength of failure against failure time; stress σ against $0.25\ \varepsilon_c$ will be obtained from $E_{s,0.25\varepsilon_c}$; and t for $0.25\ \varepsilon_c$ will be obtained from (4). The values obtained through application of these to (5) and curve fitting are given in Table 6.

Table 6

Test No	1	5	11
Mix proportion w/c	1:2:4, w/c: 37%	1:4:7, w/c: 65%	1:3, w/c: 50%
Age	4 Ws	4 Ws	4 Ws
E kg/cm²	32.1×10^4	27.2×10^4	27.5×10^4
E_1 kg/cm²	3.1×10^4	7.8×10^4	4.4×10^4
η_1 kg/cm² sec.	31.0×10^4	110.7×10^4	64.0×10^4
Retardation time η_1/E_1 sec.	10	14.2	14.5

Naturally these values do not suffice to explain the behavior in statical tests, but are able to explain behaviors within low stress in dynamical tests to a certain extent. In other words, dynamical behaviors, such as both statical and dynamical elasticity, specifically changes in dynamical elasticity under periodical stress, and the absolute value of viscosity, will be explained to some extent.

Conclusion

Thus some of the dynamical behaviors of concrete and mortar under one stroke of compressive load with the same load-velocity as earthquake load have been analyzed. It has been found that the smaller the failure time, the greater the strength of failure, and that between these two exist (1) relations.

In respect to compressibility, it is almost steady according to the quality and age of material, and regardless of failure time, there exist (2) relations.

Relations between elasticity and failure time are signified by (3), in which the smaller failure time, the greater elasticity.

From these, the coefficient of elasticity and coefficient of viscosity of a simple three-element model can be obtained, and with them, dynamical behaviors within low stress can be explained to some extent.

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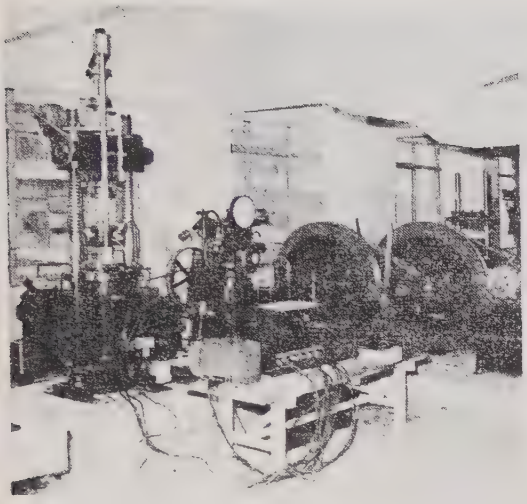


Photo. 1

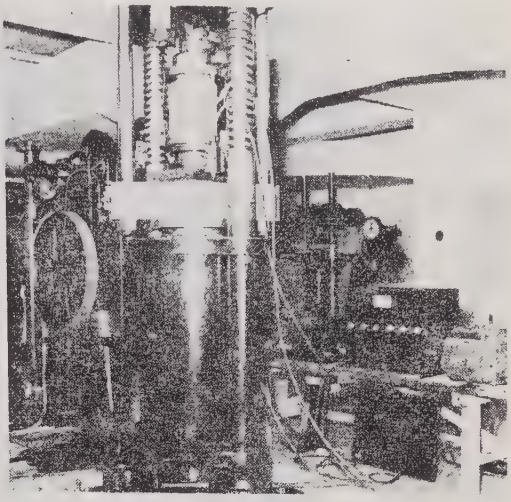


Photo. 2

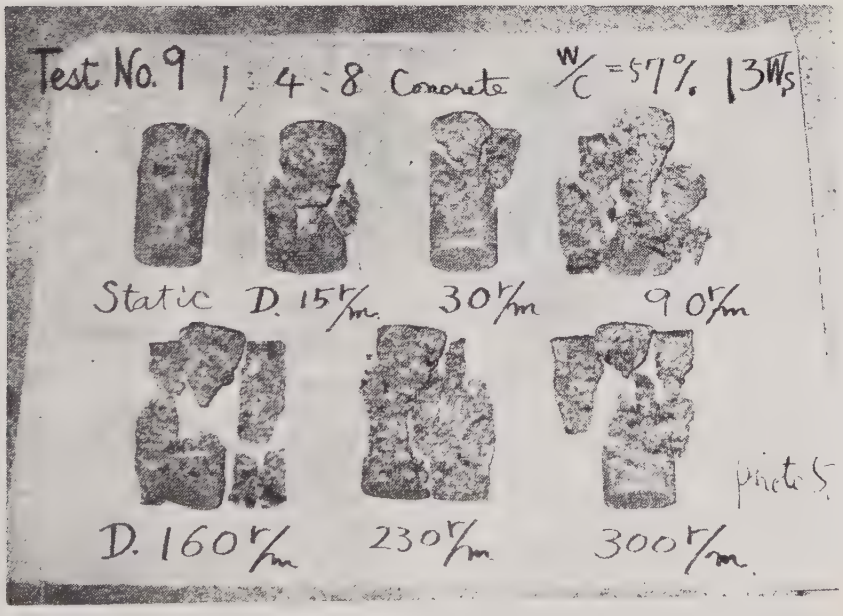
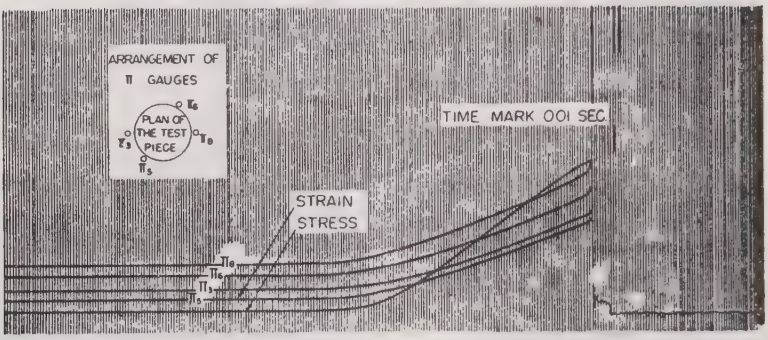


Photo. 3

FIG 1-1 TEST RECORD TEST No2-4



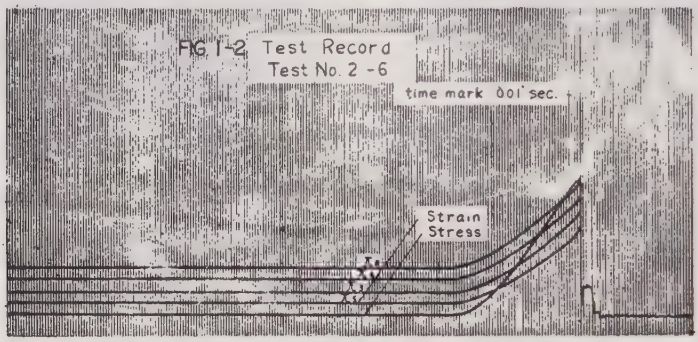


FIG. 1-3 TEST RECORD TEST No.2-9

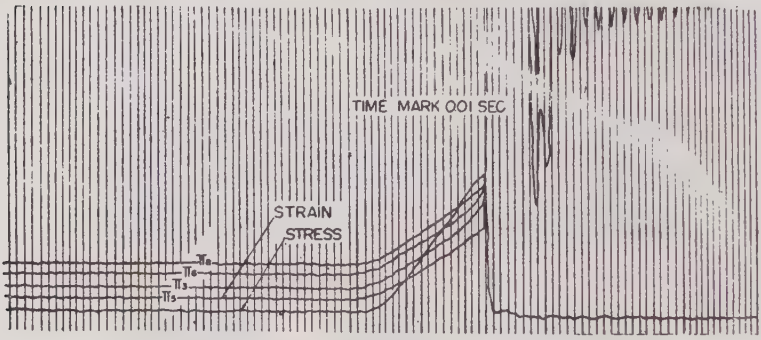
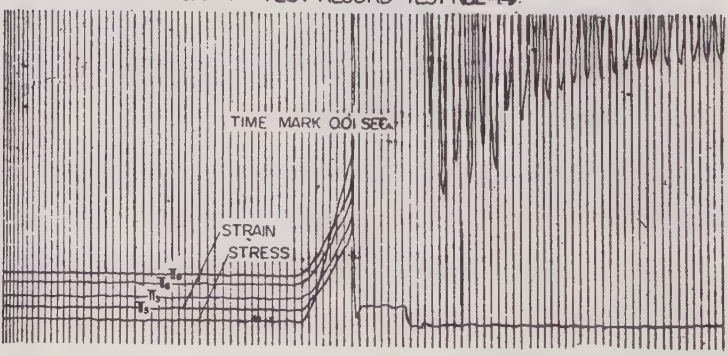
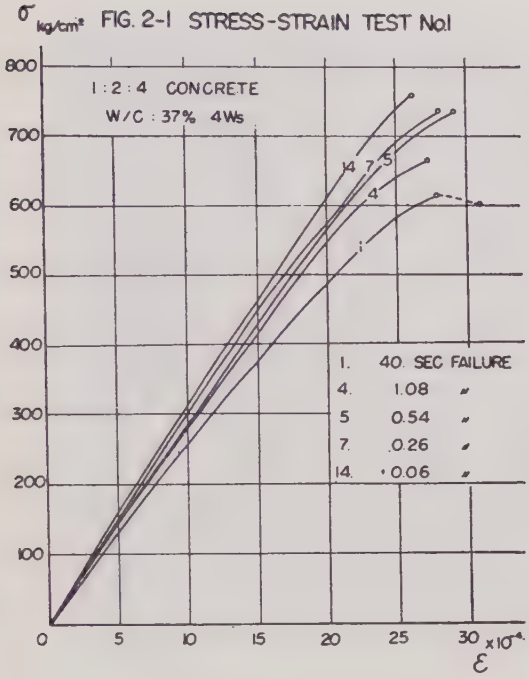
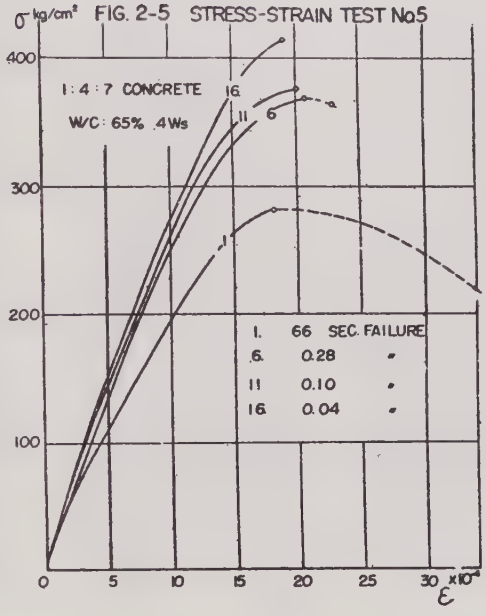
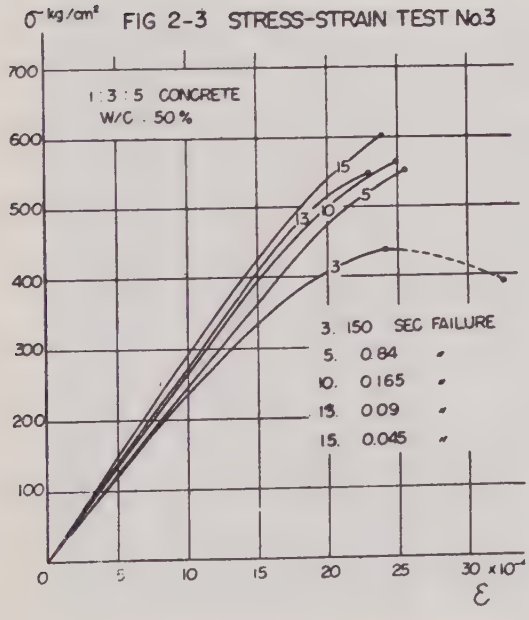


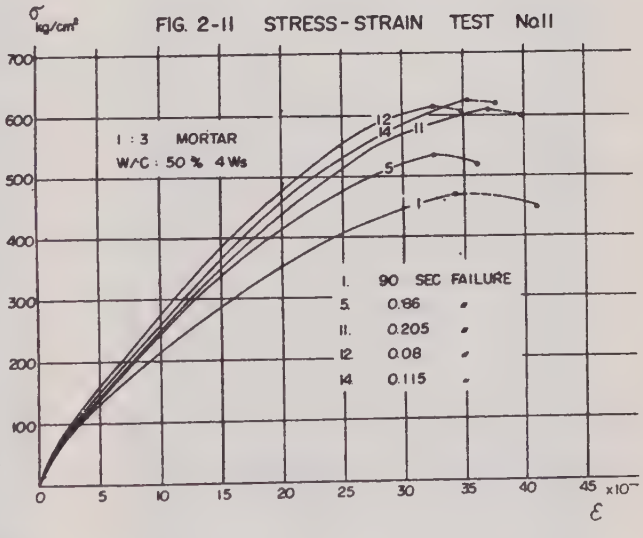
FIG. 1-4 TEST RECORD TEST No2-14.




σ kg/cm²
FIG. 2-1 STRESS-STRAIN TEST No.1
1 : 2 : 4 CONCRETE
W/C : 37% 4Ws
14
7.5
4
1
1. 40. SEC FAILURE
4. 1.08 "
5 0.54 "
7. 0.26 "
14. 0.06 "
800
700
600
500
400
300
200
100
0
5
10
15
20
25
30 x10⁻⁴
ε


σ kg/cm²
FIG. 2-5 STRESS-STRAIN TEST No.5
1 : 4 : 7 CONCRETE
W/C : 65% 4Ws
16
11
6
1
1. 66 SEC. FAILURE
6. 0.28 "
11 0.10 "
16. 0.04 "
400
300
200
100
0
5
10
15
20
25
30 x10⁻⁴
ε


σ kg/cm²
FIG 2-3 STRESS-STRAIN TEST No.3
1 : 3 : 5 CONCRETE
W/C . 50 %
15
13
10
5
3
3. 150 SEC FAILURE
5. 0.84 "
10. 0.165 "
13. 0.09 "
15. 0.045 "
700
600
500
400
300
200
100
0
5
10
15
20
25
30 x10⁻⁴
ε


σ kg/cm²
FIG. 2-11 STRESS-STRAIN TEST No.11
1 : 3 MORTAR
W/C : 50 % 4 Ws
12
14
11
5
1
1. 90 SEC FAILURE
5. 0.86 "
11. 0.205 "
12. 0.08 "
14 0.115 "
700
600
500
400
300
200
100
0
5
10
15
20
25
30
35
40
45 x10⁻⁴
ε

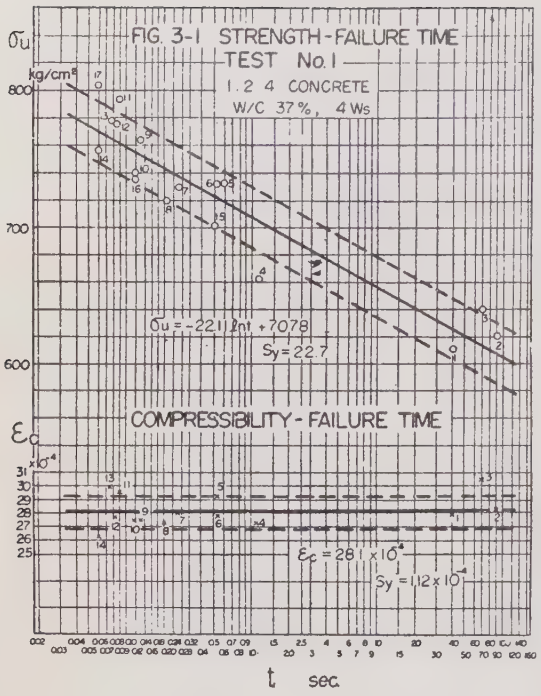


FIG. 3-1 STRENGTH-FAILURE TIME
TEST No.1
1 2 4 CONCRETE
W/C 37%, 4Ws
σu
kg/cm²
σu = -221 ℓnt + 7078
Sy = 22.7
COMPRESSIBILITY-FAILURE TIME
εc
εc = 281 x 10⁻⁴
Sy = 1.12 x 10⁻⁴
t sec.

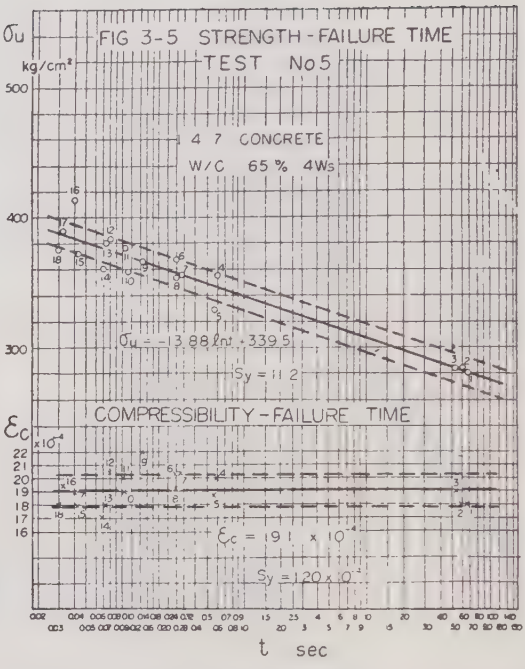


FIG 3-5 STRENGTH-FAILURE TIME
TEST No5
1 4 7 CONCRETE
W/C 65% 4Ws
σu
kg/cm²
σu = -13.88 ℓnt + 339.5
Sy = 11.2
COMPRESSIBILITY-FAILURE TIME
εc
εc = 191 x 10⁻⁴
Sy = 1.20 x 10⁻⁴
t sec

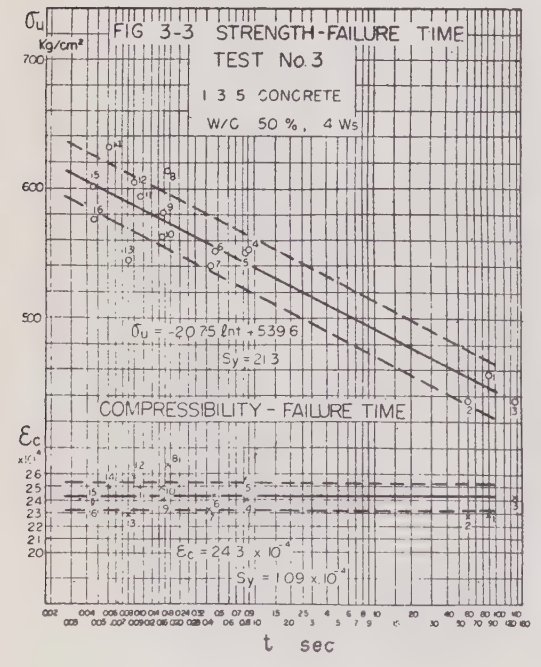


FIG 3-3 STRENGTH-FAILURE TIME
TEST No.3
1 3 5 CONCRETE
W/C 50%, 4 Ws
σu
kg/cm²
σu = -207.5 ℓnt + 539.6
Sy = 21.3
COMPRESSIBILITY-FAILURE TIME
εc
εc = 243 x 10⁻⁴
Sy = 1.09 x 10⁻⁴
t sec

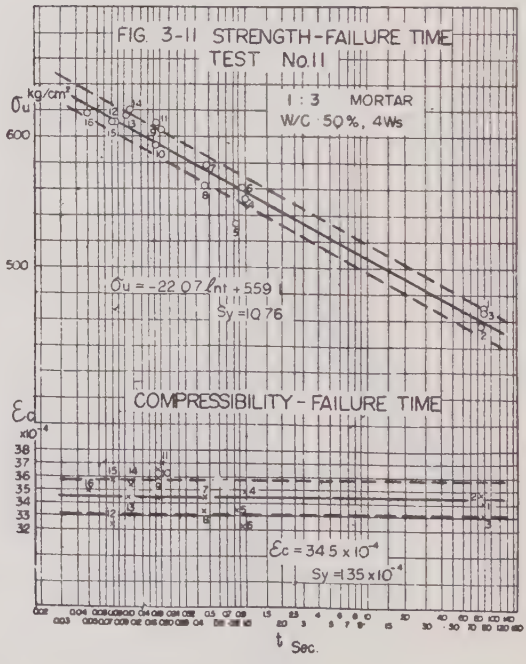


FIG. 3-11 STRENGTH-FAILURE TIME
TEST No.11
1 : 3 MORTAR
W/C 50%, 4Ws
σu
kg/cm²
σu = -22.07 ℓnt + 559.1
Sy = 10.76
COMPRESSIBILITY-FAILURE TIME
εd
εc = 345 x 10⁻⁴
Sy = 1.35 x 10⁻⁴
t Sec.

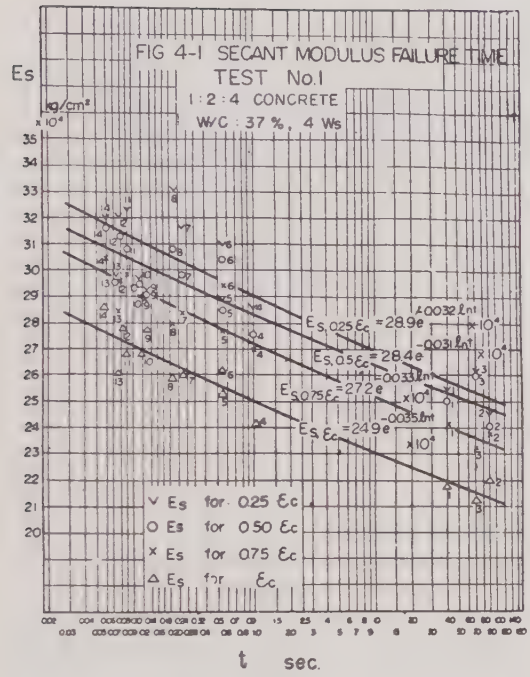


FIG 4-1 SECANT MODULUS FAILURE TIME
TEST No.1
1:2:4 CONCRETE
W/C : 37 %, 4 Ws
Es
kg/cm²
x10⁴
Es,025εc =28.9e -0032 ℓnt
Es,05εc =28.4e -0031 ℓnt
Es,075εc =27.2e -0033 ℓnt
Es,εc =24.9e -0035 ℓnt
v Es for 025 εc
O Es for 050 εc
x Es for 075 εc
△ Es for εc
t sec.

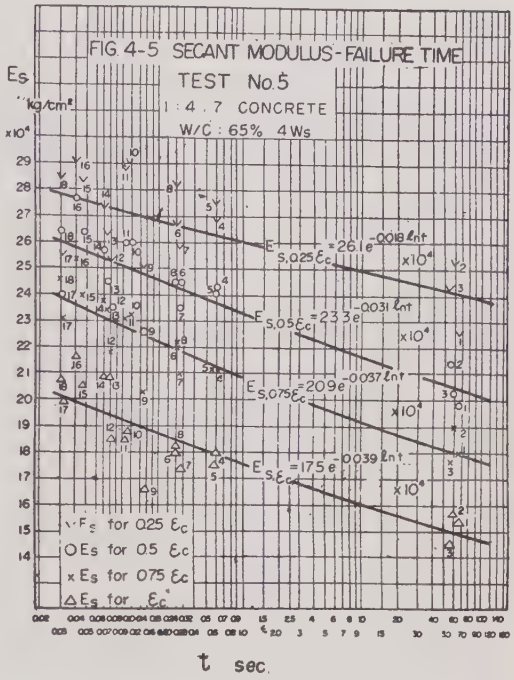


FIG 4-5 SECANT MODULUS-FAILURE TIME
TEST No.5
1:4.7 CONCRETE
W/C : 65% 4Ws
Es
kg/cm²
x10⁴
Es,025εc =26.1e -0018 ℓnt
Es,05εc =23.3e -0031 ℓnt
Es,075εc =20.9e -0037 ℓnt
Es,εc =17.5e -0039 ℓnt
v Es for 025 εc
O Es for 05 εc
x Es for 075 εc
△ Es for εc
t sec.

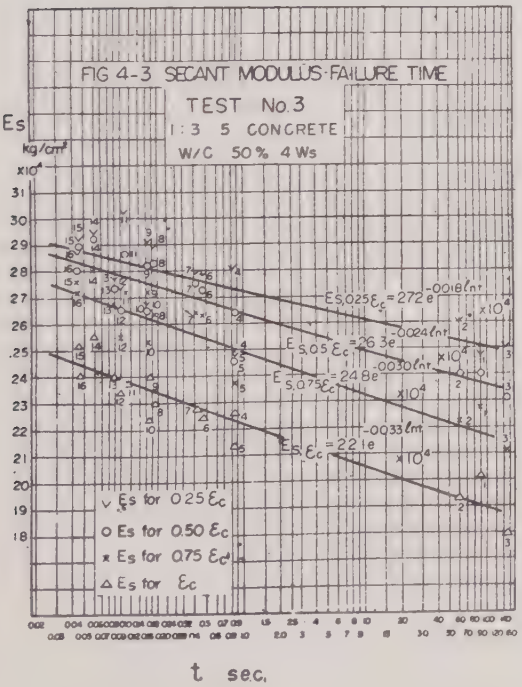


FIG 4-3 SECANT MODULUS-FAILURE TIME
TEST No.3
1:3 5 CONCRETE
W/C 50% 4Ws
Es
kg/cm²
x10⁴
Es,025εc =27.2e -0018 ℓnt
Es,05εc =26.3e -0024 ℓnt
Es,075εc =24.8e -0030 ℓnt
Es,εc =22.1e -0033 ℓnt
v Es for 025 εc
O Es for 0.50 εc
x Es for 075 εc
△ Es for εc
t sec.

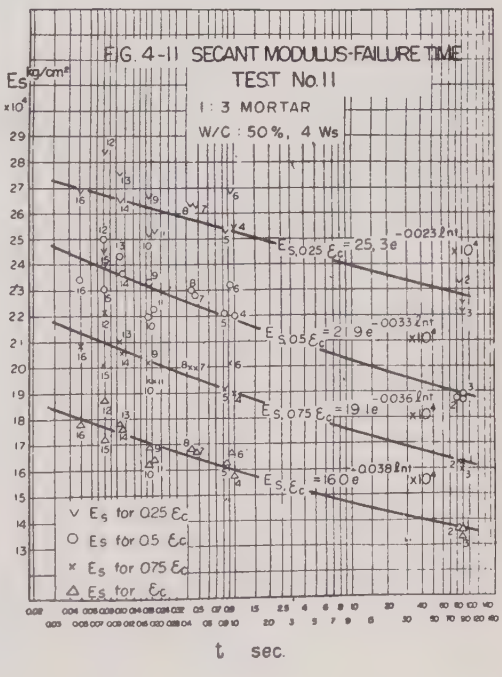


FIG 4-11 SECANT MODULUS-FAILURE TIME
TEST No.11
1:3 MORTAR
W/C : 50%, 4 Ws
Es
kg/cm²
x10⁴
Es,025εc =25.3e -0023 ℓnt
Es,05εc =21.9e -0033 ℓnt
Es,075εc =19.1e -0036 ℓnt
Es,εc =16.0e -0038 ℓnt
v Es for 025 εc
O Es for 05 εc
x Es for 075 εc
△ Es for εc
t sec.

Table - 1

Properties of the Cement

1)

Ig. loss	Ins. R	SiO_2	Al_2O_3	Fe_2O_3	Cao	Mgo	SO_3
0.75	0.27	23.21	4.03	4.13	63.76	0.93	1.98

C_3S	C_2S	C_3A	C_4AF
44.4	33.1	3.7	12.6

2)

Sp. gr.	finess
3.20	3250 cm^2/g

3)

Strength of Standard Mortar			
flow	3 day	7 day	28 day
256 m.m.	127 kg/cm^2	170	405

Table - 2

Properties of the Aggregates

Sp.gr.	Max size	Gradation	
Sagami River Natural Agg.	2.75	25 m.m.	Gravel: 25 - 20 m.m. 25%; 20 - 15 25; 15 - 10 25; 10 - 5 25
			Sand: 5 - 2.5 m.m 15%; 2.5 - 1.2 15; 1.2 - 0.6 25; 0.6 - 0.3 25; 0.3 - 0.15 15; 0.5 - 5

Table - 3

Tested Concrete & Mortar

Test No.	Age in Weeks	Mix proportion					Slump & air		
		Cement (kg/m^3)	Water (kg/m^3)	Sand (kg/m^3)	Gravel (kg/m^3)	W/c (%)	Slump (cm)	Air(%)	cement void ratio C/V
1	4	380	140	698	1296	37	0.6	1.69	0.757
2	4	380	160	658	1222	42	10.0	0.98	0.700
3	4	300	150	710	1264	50	10.3	1.18	0.580
4	4	220	125	800	1300	57	0.5	2.20	0.467
5	4	220	143	781	1276	65	3.5	1.64	0.431
6	13	380	160	658	1222	42	10.0	1.25	0.685
7	13	300	135	724	1288	45	1.5	1.68	0.618
8	13	300	150	710	1264	50	4.3	1.34	0.574
9	13	220	130	800	1300	57	0.5	2.80	0.435
10	13	220	143	781	1276	65	3.4	2.03	0.421
11	4	511	256	1533	-	50	flow 17.0	4.41	0.528
12	4	410	246	1640	-	60	16.2	5.81	0.421
13	13	511	256	1533	-	50	17.1	4.11	0.538
14	13	410	246	1640	-	60	15.3	5.79	0.422

Table - 4

Strength & Compressibility

Test No.	Relation between Strength & failure time (sec. kg/cm²)	Standard deviation Sy(kg/cm²)	Strength kg/cm²			Strength ratio σ_u100: σ_u 1:σ_u0.03	Compressibility x10^{-4}	Standard deviation Sy x10^{-4}
			σ_u 100sec.	σ_u 1 sec.	σ_u 0.03sec.			
1	σ_u = -22.11 lnt + 707.8	22.7	605.8	707.8	785.3	1:1.17:1.30	28.1	1.12
2	σ_u = -25.23 lnt + 621.8	29.0	505.5	621.8	710.3	1:1.23:1.41	23.8	1.27
3	σ_u = -20.75 lnt + 539.6	21.3	444.1	539.6	612.2	1:1.21:1.38	24.3	1.09
4	σ_u = -17.98 lnt + 405.8	14.4	325.8	405.8	468.8	1:1.25:1.44	21.2	0.99
5	σ_u = -13.88 lnt + 339.5	11.2	275.5	339.5	388.1	1:1.23:1.41	19.1	1.20
6	σ_u = -29.63 lnt + 728.3	27.4	591.7	728.3	832.8	1:1.23:1.40	26.7	1.74
7	σ_u = -31.06 lnt + 638.8	24.1	495.7	638.8	747.8	1:1.29:1.51	26.4	1.11
8	σ_u = -28.62 lnt + 604.1	27.1	472.1	604.1	704.5	1:1.28:1.49	26.9	1.13
9	σ_u = -20.96 lnt + 420.3	16.3	323.8	420.3	503.7	1:1.30:1.56	21.7	1.20
10	σ_u = -15.31 lnt + 353.1	11.6	282.5	353.1	406.8	1:1.25:1.44	19.8	1.34
11	σ_u = -22.07 lnt + 559.1	10.7	457.3	559.1	636.5	1:1.22:1.39	34.5	1.34
12	σ_u = -13.43 lnt + 309.4	7.6	247.5	309.4	356.5	1:1.25:1.44	26.2	0.89
13	σ_u = -30.54 lnt + 744.9	13.4	604.2	744.9	851.9	1:1.23:1.41	38.2	1.01
14	σ_u = -21.78 lnt + 498.1	10.2	397.9	498.1	574.4	1:1.25:1.44	32.5	1.08

Table 5 - 1

Secant Modulus (1)

Test No.	Relation between (sec. kg/cm²) Secant Modulus for 0.25 ε_c & failure time x10⁴	Standard deviation x10⁴ (kg/cm²)	Relation between (sec. kg/cm²) Secant Modulus for 0.5 ε_c & failure time x10⁴	Standard deviation x10⁴ (kg/cm²)
1	$E_{s,0.25\varepsilon_c} = 28.9\ e^{-0.032\ lnt}$	1.25	$E_{s,0.5\varepsilon_c} = 28.4\ e^{-0.031\ lnt}$	0.85
2	$E_{s,0.25\varepsilon_c} = 35.0\ e^{-0.027\ lnt}$	2.01	$E_{s,0.5\varepsilon_c} = 31.4\ e^{-0.030\ lnt}$	1.28
3	$E_{s,0.25\varepsilon_c} = 27.2\ e^{-0.018\ lnt}$	1.04	$E_{s,0.5\varepsilon_c} = 26.3\ e^{-0.024\ lnt}$	0.82
4	$E_{s,0.25\varepsilon_c} = 26.2\ e^{-0.025\ lnt}$	1.71	$E_{s,0.5\varepsilon_c} = 24.7\ e^{-0.030\ lnt}$	1.40
5	$E_{s,0.25\varepsilon_c} = 26.1\ e^{-0.018\ lnt}$	1.34	$E_{s,0.5\varepsilon_c} = 23.3\ e^{-0.031\ lnt}$	1.08
6	$E_{s,0.25\varepsilon_c} = 30.5\ e^{-0.038\ lnt}$	2.27	$E_{s,0.5\varepsilon_c} = 30.1\ e^{-0.039\ lnt}$	1.98
7	$E_{s,0.25\varepsilon_c} = 29.7\ e^{-0.038\ lnt}$	1.26	$E_{s,0.5\varepsilon_c} = 27.9\ e^{-0.039\ lnt}$	0.90
8	$E_{s,0.25\varepsilon_c} = 26.6\ e^{-0.046\ lnt}$	1.40	$E_{s,0.5\varepsilon_c} = 25.5\ e^{-0.045\ lnt}$	1.20
9	$E_{s,0.25\varepsilon_c} = 29.4\ e^{-0.030\ lnt}$	1.17	$E_{s,0.5\varepsilon_c} = 25.4\ e^{-0.034\ lnt}$	0.89
10	$E_{s,0.25\varepsilon_c} = 26.8\ e^{-0.035\ lnt}$	1.39	$E_{s,0.5\varepsilon_c} = 23.1\ e^{-0.044\ lnt}$	1.03
11	$E_{s,0.25\varepsilon_c} = 25.3\ e^{-0.023\ lnt}$	0.95	$E_{s,0.5\varepsilon_c} = 21.9\ e^{-0.033\ lnt}$	0.67
12	$E_{s,0.25\varepsilon_c} = 22.5\ e^{-0.027\ lnt}$	1.11	$E_{s,0.5\varepsilon_c} = 17.9\ e^{-0.031\ lnt}$	0.59
13	$E_{s,0.25\varepsilon_c} = 28.4\ e^{-0.039\ lnt}$	1.27	$E_{s,0.5\varepsilon_c} = 25.4\ e^{-0.039\ lnt}$	0.88
14	$E_{s,0.25\varepsilon_c} = 25.7\ e^{-0.023\ lnt}$	0.62	$E_{s,0.5\varepsilon_c} = 21.3\ e^{-0.029\ lnt}$	0.50

Table 5 - 2

Secant Modulus (2)

Test No.	Relation between (sec. kg/cm²) Secant Modulus for 0.75 ε_c & failure time x10⁴	Standard deviation x10⁴ (kg/cm²)	Relation between (sec. kg/cm²) Secant Modulus for ε_c & failure time x10⁴	Standard deviation x10⁴ (kg/cm²)
1	$E_{s,0.75\varepsilon c} = 27.2\ e^{-0.033\ lnt}$	0.62	$E_{s,\varepsilon c} = 24.9\ e^{-0.035\ lnt}$	0.65
2	$E_{s,0.75\varepsilon c} = 29.1\ e^{-0.037\ lnt}$	1.25	$E_{s,\varepsilon c} = 25.9\ e^{-0.039\ lnt}$	1.13
3	$E_{s,0.75\varepsilon c} = 24.8\ e^{-0.030\ lnt}$	0.77	$E_{s,\varepsilon c} = 22.1\ e^{-0.033\ lnt}$	0.67
4	$E_{s,0.75\varepsilon c} = 22.3\ e^{-0.039\ lnt}$	1.06	$E_{s,\varepsilon c} = 18.7\ e^{-0.054\ lnt}$	1.04
5	$E_{s,0.75\varepsilon c} = 20.9\ e^{-0.037\ lnt}$	0.88	$E_{s,\varepsilon c} = 17.5\ e^{-0.039\ lnt}$	0.99
6	$E_{s,0.75\varepsilon c} = 29.7\ e^{-0.040\ lnt}$	1.89	$E_{s,\varepsilon c} = 27.1\ e^{-0.041\ lnt}$	1.79
7	$E_{s,0.75\varepsilon c} = 26.7\ e^{-0.039\ lnt}$	0.86	$E_{s,\varepsilon c} = 24.1\ e^{-0.044\ lnt}$	1.01
8	$E_{s,0.75\varepsilon c} = 24.8\ e^{-0.045\ lnt}$	1.06	$E_{s,\varepsilon c} = 22.3\ e^{-0.046\ lnt}$	1.00
9	$E_{s,0.75\varepsilon c} = 22.5\ e^{-0.043\ lnt}$	1.04	$E_{s,\varepsilon c} = 19.0\ e^{-0.052\ lnt}$	1.03
10	$E_{s,0.75\varepsilon c} = 20.7\ e^{-0.040\ lnt}$	0.72	$E_{s,\varepsilon c} = 17.7\ e^{-0.044\ lnt}$	0.85
11	$E_{s,0.75\varepsilon c} = 19.1\ e^{-0.036\ lnt}$	0.55	$E_{s,\varepsilon c} = 16.0\ e^{-0.038\ lnt}$	0.46
12	$E_{s,0.75\varepsilon c} = 14.7\ e^{-0.035\ lnt}$	0.44	$E_{s,\varepsilon c} = 11.7\ e^{-0.038\ lnt}$	0.35
13	$E_{s,0.75\varepsilon c} = 22.4\ e^{-0.040\ lnt}$	0.81	$E_{s,\varepsilon c} = 19.4\ e^{-0.040\ lnt}$	0.63
14	$E_{s,0.75\varepsilon c} = 18.2\ e^{-0.034\ lnt}$	0.44	$E_{s,\varepsilon c} = 15.1\ e^{-0.041\ lnt}$	0.48

BEHAVIOUR OF PLAIN CONCRETE UNDER DYNAMIC LOADING WITH STRAINING RATE COMPARABLE TO EARTHQUAKE LOADING

by S. Ban* and H. Muguruma**

Introduction

In designing a structure subject to violent earthquake load, the engineer faces the twofold problem in determining the peak loads on the structure. At present, there are many analytical researches on the vibration of structures under the action of violent earthquakes. However, these results cannot be applied reasonably to the practical design of the actual structures because of their assumptions. Among those, there is a misunderstanding regarding with the restoration force-displacement relations actually shown by structure under dynamic load, i.e., the ideal elasto-plastic or the rigid-plastic assumption.

Although the analysis of the structures based on such ideal bodies is, of course, necessary to clarify the fundamental behaviours of structure under the action of seismic waves, it is also required to improve the assumptions on the restoration force-displacement relations in the actual structures, because they will be necessary to extend the analytical results obtained on the ideal body to the more reasonable design of actual structures.

Since the restoration force-displacement relations of structures is depending on their composing members or materials, the behaviour of members or materials under the dynamic load should be studied precisely in advance. While a number of investigators has studied the properties of structural members, i.e., beams and columns, under the impact or repeated loading, the test results obtained did not agree so well with the analysis as were expected. It will be evident that the behaviour of members under the dynamic load is decided by the stress-strain relations of structural materials under similar dynamic loading. From this concept, it can be concluded that the fundamental problem is to study the behaviour of structural materials, especially their stress-strain relationships and the strength, under the dynamic load.

In this study, it was the purpose to investigate the behaviours of plain concrete under the impact or repeated loading with loading rate comparable to the earthquake forces.

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Scope of the Tests

A number of investigators has studied the properties of plain concrete under impact or repeated loading. D. A. Abrams (1), P. G. Jones and F. E. Richart (2) employed relatively low loading speeds, because the aim of their studies was to designate the standard method of compression tests for concrete. D. Watstein (3), C. Katsuta (4), J. Takeda (5), D. McHenry and J. J. Shiderer (6) had tested a plain concrete in compression at the rates of strain varying from 10^{-5} to 10 in./in. sec.. Considering the vibration of the ordinary reinforced concrete structures, its frequency is expected to be 0.5 to 10 cycle per sec. at the time of earthquake. Therefore it is desirable to consider the impact whose duration is about 1 to 0.05 sec.. The corresponding average rate of strain can be predicted as 3×10^{-3} to 6×10^{-2} in./in.sec.. Therefore, in this study the average duration of impact load was selected as 1 sec. throughout the impact loading tests.

The repeated loading tests were also undertaken in this study. On the fatigue behaviour of plain concrete many test results were already presented (7). However, almost of them were to investigate the physical properties of concrete for the permanent use of structures under the usual design loads, and therefore, the main object was to determine the fatigue limits of concrete as a basis of safety design for reinforced concrete structures under the stress of several millions repetition. From the standpoint of the analysis of structure against the violent earthquake forces, it is consequently important to confirm the stress-strain relationship of plain concrete under the violently repeated stress, where a high speed repetition and a large number of cycles are not necessary but the upper limit of stress should be taken higher than that of usual repeated tests. Therefore, in this report the dynamic stress-strain relations were investigated under the repeated stress, the upper limit being 60 to 90 % of static strength.

Program of the Experiments

(1) Test specimens

The study included the concretes having the different mix proportions as listed in Table 1. Both static and dynamic (impact or repeated) loading specimens were prepared in each mix of concrete.

The materials used in the concrete specimens were normal portland cement and river aggregates under 15 m.m..

The test specimens used in Dynamic and companion static tests were 7.5 x 15 cm cylinders. The concrete was mixed by hand and cast into the steel molds by the method of tapping with 6 m.m. dia. steel bar. At the age of 3 days, the mold was removed and then the specimens were partly moist-cured and partly cured in air. Prior to testing the moist-cured specimens were air-dryed for 3 days to get the surface of concrete dry enough for application of bonded type wire strain gages.

(2) Test procedure and apparatus

(a) Static tests ----- The 7.5 x 15 cm companion cylinders were tested in a 100 ton capacity hydraulic machine. The load was applied to the specimen at a loading rate of about 20 kg/cm^2 sec.. The loading speed was adjusted by controlling the valve in oil pipe at the initial stage of loading and no further adjustments were made until the specimen began to yield before the maximum load was reached.

The bonded type wire strain gauges were used for the compressive strain measurement. A pair of gauges mounted on opposite sides at mid-height of each specimen were connected in parallel and the average of the output of each gauge was recorded directly.

(b) Impact loading tests ----- Tests were made in a loading apparatus composed of two levers shown in Fig. 1. The 7.5 x 15 cm concrete specimen was placed on a 25 ton capacity load cell fixed on the lower joist. The impact load was applied on the specimen by slowly dropping a 500kg dead weight on extreme end of upper lever. The strain was measured in the similar method as described before. The outputs of the gauges mounted on the specimen and the load cell were fed into the dynamic strain indicator with wide band strain amplifier and self-recording oscillograph as shown in Fig. 2. A block diagram of equipment is shown in Fig. 3.

(c) Repeated loading tests ----- The fatigue testing machine specially prepared for the fatigue tests under the low speed loading cycles was used. The speed of loading cycles was 1.2 cycle per sec., and was kept constant throughout the tests. The upper limit of loading cycles was selected in the range from 60 to 90 % of the static breaking load of the same concrete cylinder. This adjustment was done by mechanical cams which move the base plate up and down. The compressive strain of concrete specimens as well as the corresponding load were recorded by the same system as that in the impact loading tests. The fatigue testing machine and measuring equipments used for the tests are shown in Fig. 4.

Results of Impact Loading Tests and Discussions

(1) Stress and strain of concrete

Typical records of stress or strain versus time duration are illustrated in Fig. 5. In Fig. 5, it will be noted that the rate of loading was gradually increased at the first stage of loading (see the portion AB in Fig. 5) and was contrarily decreased at the last stage (see the portion CD in Fig. 5). At the intermediate stage the rate of loading was kept almost unvaried (see the portion BC in Fig. 5). It has been verified by the many past investigations that these gradual changes of loading rate have no influence upon the properties of concrete under impact loading.

Typical stress-strain relations obtained from the stress-time and strain-time records are shown in Fig. 6. From those the initial tangent modulus and the secant modulus corresponding to the stress level of one-third of dynamic compressive strength were calculated.

(2) Comparison between the static and impact loading test results

The properties of concrete under the static and impact loadings are compared in Table 2, in which the average values obtained for each concrete mix are listed. The ratios of the dynamic (impact loading) to static compressive strengths, secant moduli of elasticity, maximum compressive strains and strains corresponding to the maximum stress are given in Table 2 and are shown in Fig. 7 (a), (b) and (c).

From Fig. 7 (a) it will be seen that the effect of the dynamic loading upon the compressive strength is more remarkable for the weaker concrete than the stronger concrete. However, no significant difference in strength is recognized in the concrete having the static strength more than 260 kg/cm^2 . Similar properties were found out in the comparison between dynamic and static secant modulus of elasticity (see Fig. 7 (b)). On the contrary, the maximum compressive strain was observed smaller in the impact loading tests than those in the static tests. The decrease in the maximum compressive strain was also found out larger with the decrease in static strength, provided that the static strength is less than 260 kg/cm^2 .

(3) Assumption of stress-strain curve of concrete under impact loading

As mentioned above, the impact loading with the loading rate from 200 to 400 kg/cm^2 sec. had considerable influences upon the properties of concrete having the static compressive strength less than 260 kg/cm^2 The variation of secant modulus of elasticity as well as maximum compressive strain will not be neglected in the theoretical analysis of the elastic and plastic behaviours of reinforced concrete members under seismic action (8).

Under the assumption of ideal stress-strain curve shown in Fig. 8 The dynamic stress-strain relation will be predicted through the static one, provided the ratios of dynamic to static compressive strength, secant modulus of elasticity and maximum strain are given as follows:

For concrete having the static strength less than 260 kg/cm^2

$$\frac{E_{dc}}{E_{sc}} = 1.975 - 0.00375\,\sigma_{sc} \qquad (1)$$

$$\frac{\sigma_{dc}}{\sigma_{sc}} = 2.3 - 0.005\,\sigma_{sc} \qquad (2)$$

$$\frac{\varepsilon_{dc}}{\varepsilon_{sc}} = -0.3 + 0.005\,\sigma_{sc} \qquad (3)$$

For concrete having the static strength more than 260 kg/cm^2

$$\frac{E_{dc}}{E_{sc}} = \frac{\sigma_{dc}}{\sigma_{sc}} = \frac{\varepsilon_{dc}}{\varepsilon_{sc}} = 1 \qquad (4)$$

Results of Repeated Loading Tests and Discussions

(1) Static test results

The compressive strength obtained from the static tests of companion cylinders are summarized in Table 1 (b).

(2) Stress-strain curves under repeated loading

Typical stress-strain relations measured by the dynamic strain indicator during the repeated loading are shown in Fig. 9 with commentary on the initial strain and upper limit stress ratio. This illustrates that the stress-strain curve varies with the number of load repetitions. Whatever the ratio of upper limit to static strength may be large or small, the convex upward curve straightens under the loading cycles and finally become concave upward prior to the failure. And this shows that concrete possesses a property similar to strain hardening in metals. The degree of concavity is expected to warn the failure. As the upper limit of loading cycles becomes larger, the rapid transition in curves from convex upward to concave upward was recognized in the tests. These transitions had been also observed in the fatigue tests under the high speed loading cycles with relatively small stress intensity less than 60 % of the compressive strength (7).

Such property as strain hardening in stress-strain relations of plain concrete is of particular importance to discuss theoretically the elastic and plastic behaviours of actual reinforced concrete members or structures subject to a high repeated load. However, the curves obtained are too complicated to be expressed in simple formula. Therefore, in this study, several basic terms on stress-strain curve were representatively investigated, i.e., the increase of compressive strain at upper limit, the increase of permanent strain and the decrease of initial tangent or secant modulus of elasticity at each loading or unloading.

(3) Maximum or permanent strain versus number of loading cycles

Fig. 10 and Fig. 11 shows the maximum and permanent strains of concrete in relation to the number of loading cycles. From those, it is evident that the maximum strain as well as the permanent strain increased gradually with the loading cycles. The amount of such increases had no obvious relations to the concrete mixes or the static strength in compression, but was evidently related to the maximum compressive strain observed by the first loading. This strain will be defined as " original strain " and the total number of loading cycles before the failure occurs will be defined as " endurance number " hereafter. The larger the original strain , the endurance number became less, and on the contrary the maximum strain as well as the permanent strain at the fatigue failure became larger. Especially, when the original strain exceeded a limited value of about 0.1 %, the rate of strain increase became excessively larger, and the endurance number was about 30 or less. This can be exactly seen in Fig. 12 giving the relation between the original strain and the endurance number.

Consequently, it can be predicted that when a reinforced concrete members are subjected to the repetitive load such as the compressive fiber strain of concrete exceeds 0.1 %, the maximum and permanent displacements of the members will continue to increase rapidly and the members will result in fatigue after further loading cycles.

Additionally, a curve giving the maximum or permanent strain can be decided in relation to the endurance number and it is plotted in Fig. 10 and Fig. 11, respectively. Using Fig. 12 and Fig. 10 or Fig. 11, it is possible to predict the number of loading cycles which can be withstood by concrete, when the intensity of loading cycles as well as the maximum or permanent strain at the fatigue failure are previously assumed.

(4) Relationship between modulus of elasticity and number of loading cycles

As it can be seen in Fig. 9, the modulus of elasticity of concrete varied gradually with the increase in number of loading cycles. Simplifying the stress-strain curves at each loading cycles as shown in Fig. 13, the initial tangent moduli E_A and E_C as well as the tangent moduli E_B and E_D were calculated for each loading cycle. In fig. 14 the variations of a initial tangent modulus E_A are plotted against the number of loading cycles as an example. It can be seen from Fig. 14 that almost of the variations in the elastic modulus took place during the first 20 - 30 loading cycles and thereafter the obvious variations were not recognized. Especially, the variations were remarkable in the first 2 or 3 loading cycles. Consequently, we can conclude that the dynamic behaviour of a reinforced concrete member will change remarkably under the first 2 or 3 loading, when the member is subjected to high loading repetition.

Conclusions

From the test results following conclusions were derived.

Impact loading tests

(1) From the comparison with the static tests on plain concrete, it was found that under the impact loading with the loading rate from 200 to 400 kg/cm^2 sec. the compressive strength of plain concrete as well as the secant modulus were observed larger than those obtained from the static tests.

(2) The maximum compressive strain and the strain corresponding to the maximum stress of strain-stress relation were measured remarkably smaller by impact loading than by static loading.

(3) These differences became more distinctly when the concrete tested showed lower strength, but were not recognized at all for the concrete having the static strength more than 260 kg/cm^2

Repeated loading tests

(1) Under the repetition of high load (60 to 90 % of static crushing load) and the constant loading speed of 1.2 cycle per sec.remarkable transitions from convex upward to concave upward were recognized in the stress-strain curves. This shows that concrete possesses a property

similar to strain hardening in metals.

(2) The maximum strain increase and permanent strain increase took place in contact with the number of loading cycles. This phenomena was very remarkable when the maximum strain by the first loading exceeded 0.1 %.

(3) The initial as well as the secant modulus of elasticity varied in various ways depending on the intensity of load. However, most of these variations took place in the early stage of loading cycles, usually during the first 20 to 30 cycles.

(4) Number of cycles withstood by concrete before failure occurred was found very small under the loading cycles with the original strain exceeding 0.1 %.

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Nomenclature

E_{dc} and E_{sc} = Dynamic and static secant moduli of elasticity (in kg/cm^2)

σ_{dc} and σ_{sc} = Dynamic and static compressive strengths (in kg/cm^2)

ε_{dc} and ε_{sc} = Dynamic and static maximum strains

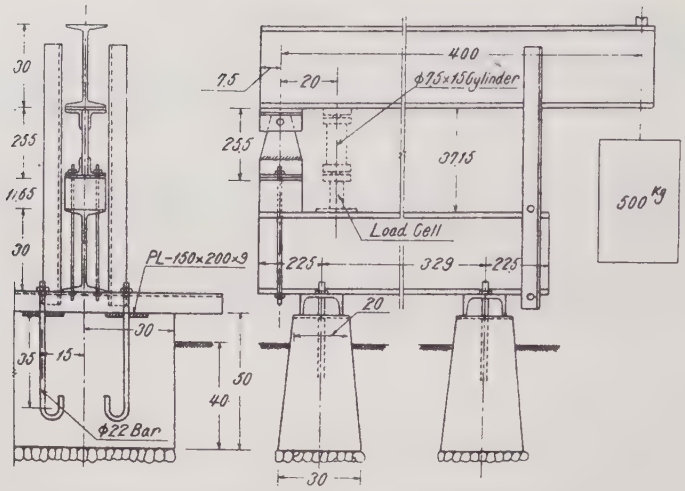


Fig. 1 Impact loading apparatus.

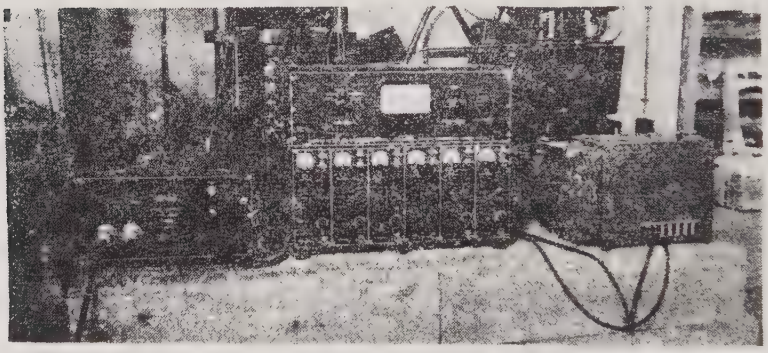


Fig. 2 Dynamic strain measuring apparatus.

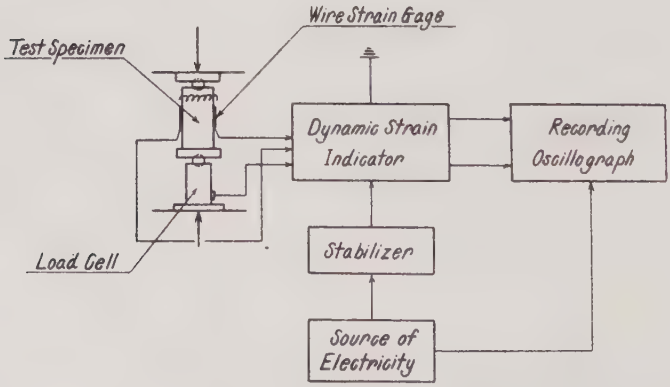


Fig. 3 Block diagram of equipment for stress and strain measurings.

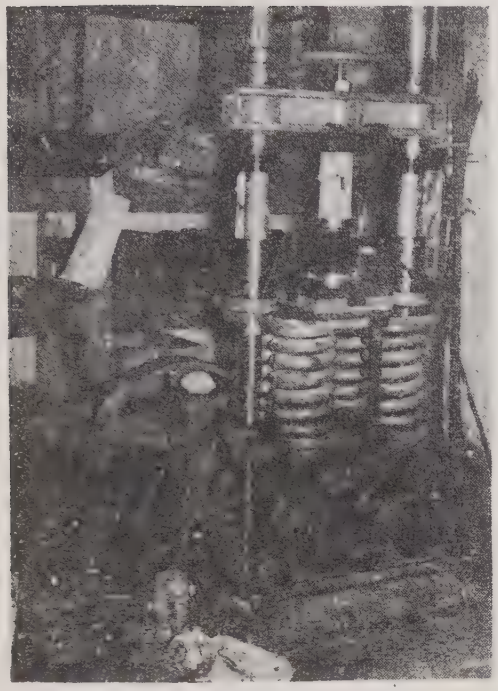


Fig. 4 Fatigue testing machine.

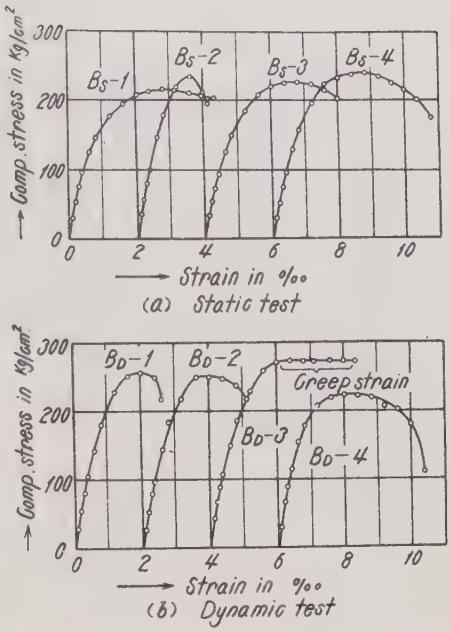


Fig. 6 Typical static and dynamic stress-strain curves obtained from the impact tests of B - Mix concrete.

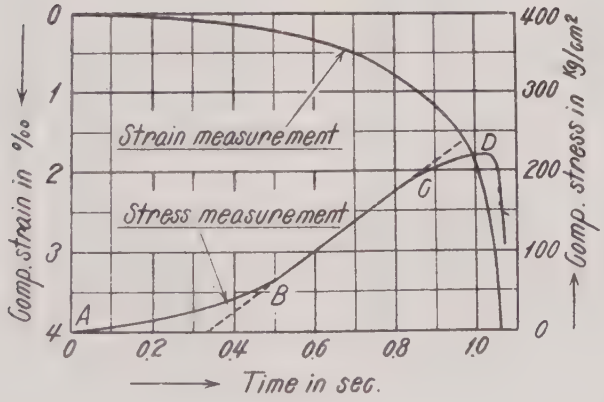


Fig. 5 Typical stress-time and strain-time records of concrete under impact loading (Specimen Bd - 4).

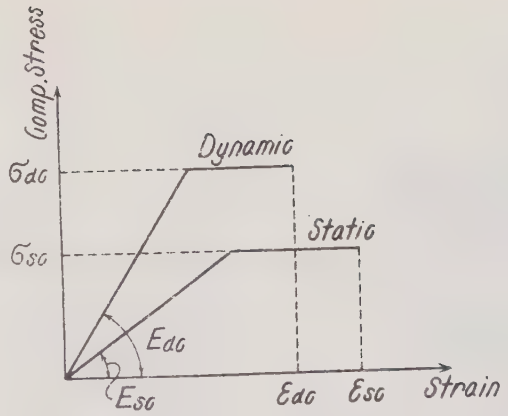


Fig. 8 Ideal stress-strain curves.

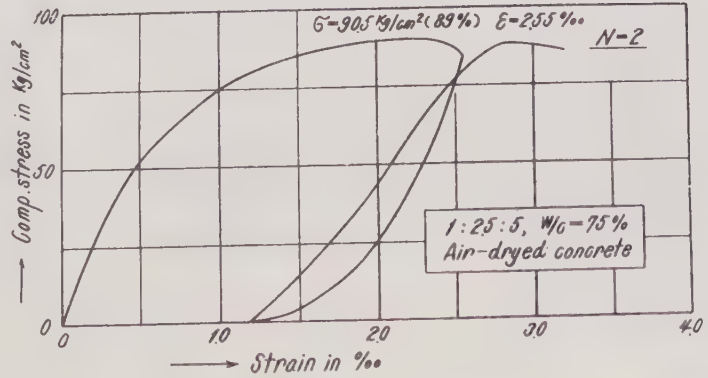


Fig. 9 Typical stress-strain curves under repetitive load.

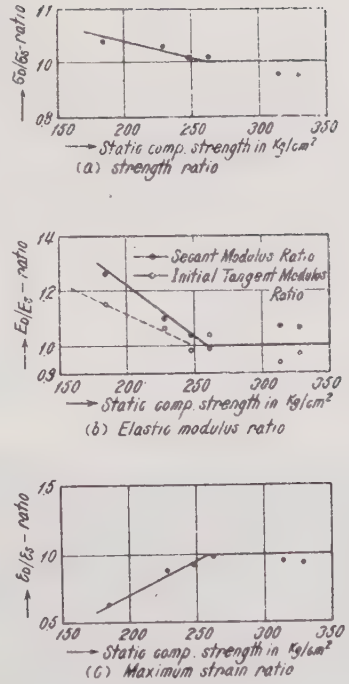


Fig. 7 Comparision between impact loading and static test results.

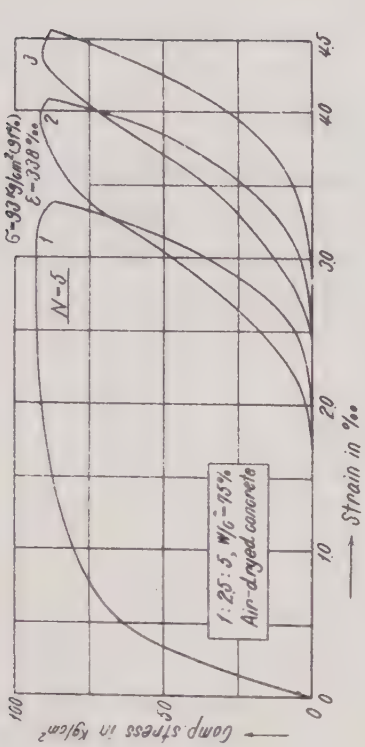


Fig. 9(b)

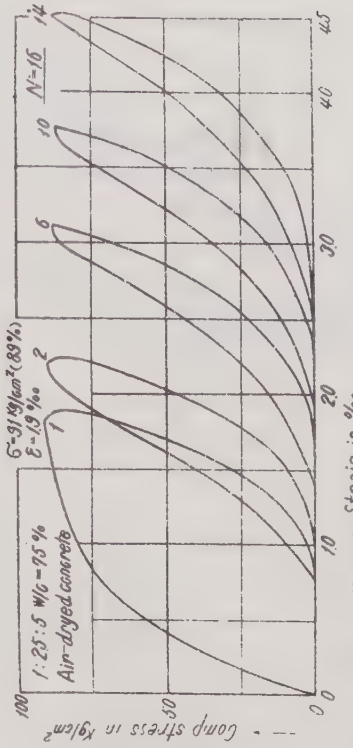


Fig. 9(c)

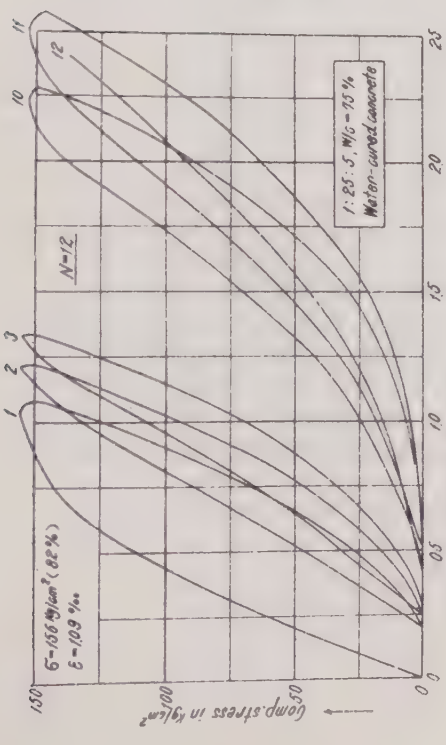


Fig. 9(d)

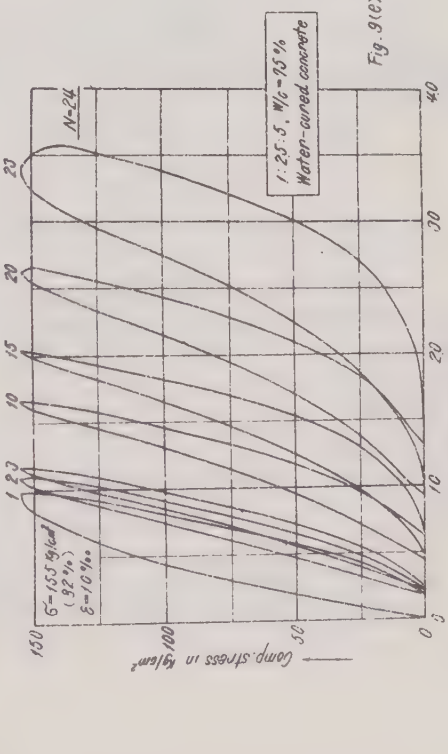


Fig. 9(e)

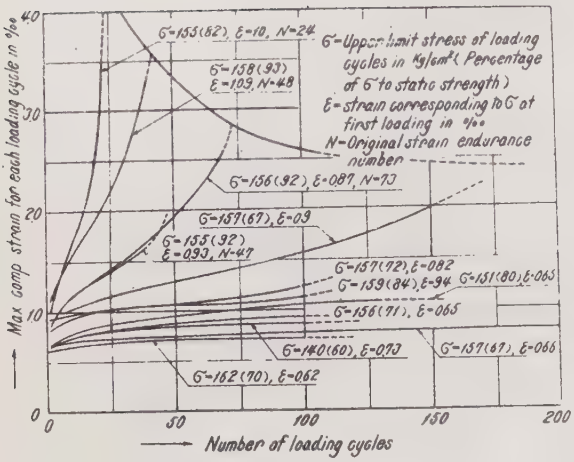

σ=155(82), ε=10, N=24
σ=158(93) ε=109, N=48
σ=156(92), ε=087, N=73
σ=157(67), ε=09
σ=155(92) ε=093, N=47
σ=157(72), ε=082
σ=159(84), ε=94
σ=151(80), ε=065
σ=156(71), ε=065
σ=140(60), ε=093
σ=157(67), ε=066
σ=162(70), ε=062
σ=Upper limit stress of loading cycles in Kg/cm² (Percentage of σ to static strength)
ε=strain corresponding to σ at first loading in ‰
N=Original strain endurance number
Max comp strain for each loading cycle in ‰
Number of loading cycles

Fig. 10

Fig. 10

Relations between maximum compressive strain for each loading cycle and number of loading cycles.

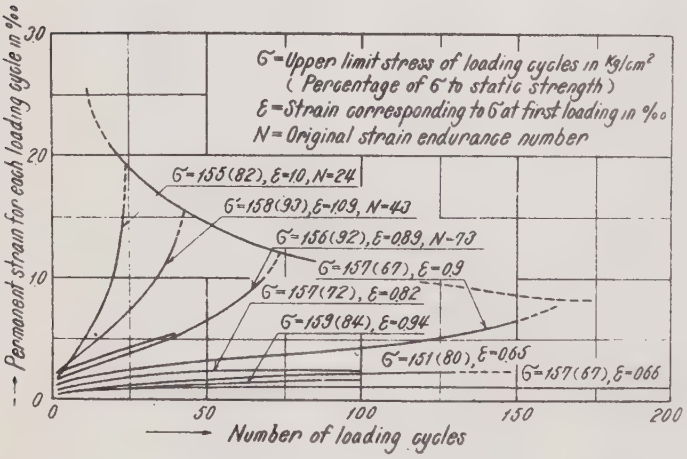

σ=Upper limit stress of loading cycles in Kg/cm² (Percentage of σ to static strength)
ε=Strain corresponding to σ at first loading in ‰
N=Original strain endurance number
σ=155(82), ε=10, N=24
σ=158(93), ε=109, N=43
σ=156(92), ε=089, N=73
σ=157(67), ε=09
σ=157(72), ε=082
σ=159(84), ε=094
σ=151(80), ε=065
σ=157(67), ε=066
Permanent strain for each loading cycle in ‰
Number of loading cycles

Fig. 11

Fig. 11

Relations between permanent strain for each loading cycle and number of loading cycles.

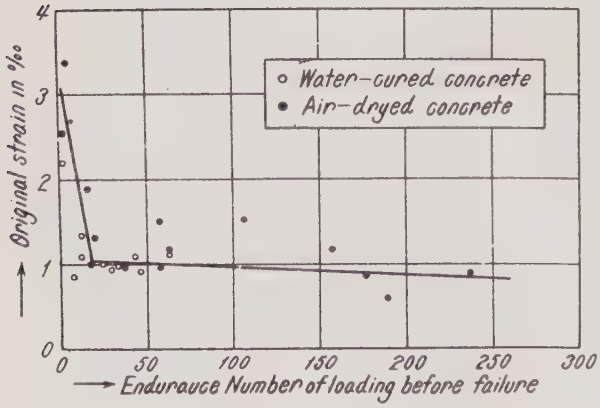


Fig. 12

Fig. 12

Relations between maximum strain at the first loading and number of loading cycles before failure occurred.

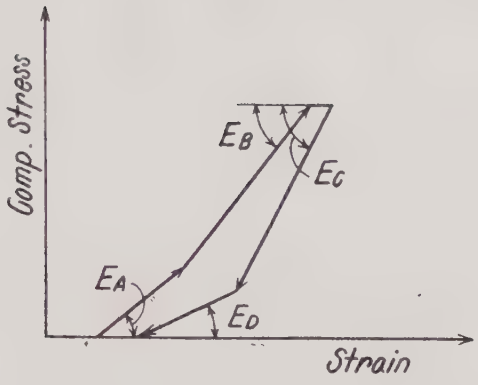


Fig. 13 Simplified stress-strain curve under repetitive load.

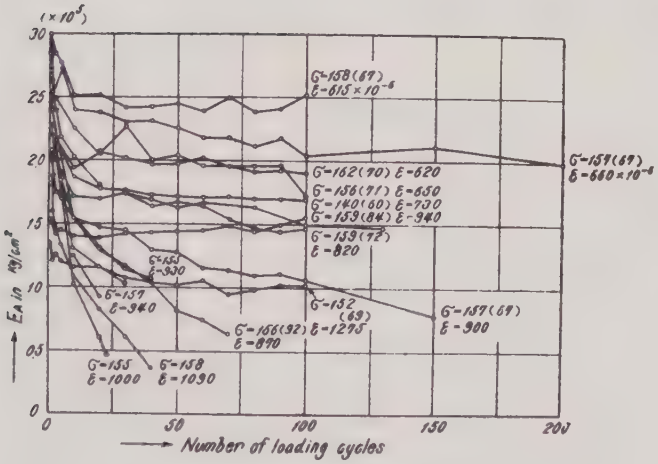


Fig. 14 Relations between E_A and number of loading cycles.

Table 1 (a) Test Schedules for Impact Loading

Test Series	Mix Proportion by Weight	W/C-Ratio in %	Test Age in Days	Number of Test Specimens	
				Static Test	Impact Loading Test
A	1 : 2.5 : 5	75	28	3	4
B	1 : 2.5 : 5	70	28	4	4
C	1 : 2 : 4	65	28	5	5
D	1 : 2 : 4	60	28	4	5
E	1 : 1.5 : 3	55	28	4	5
F	1 : 1.5 : 3	50	28	4	5

Table 1 (b) Test Schedules for Repeated Loading

Test Series	Mix Proportion by Weight	W/C-Ratio in %	Test Age in Days	Static Comp. Strength in kg/cm^2	Number of Test Specimens	
					Static Test	Repeated Loading Test
AD*	1 : 2.5 : 5	75	28	116	5	11
BD*	1 : 2.5 : 5	70	28	108	5	10
CD*	1 : 2 : 4	65	28	178	3	3
AW**	1 : 2.5 : 5	75	28	179	6	10
BW**	1 : 2.5 : 5	70	28	234	6	5
CW**	1 : 2 : 4	65	28	220	4	5

* Air-dryed concrete, ** Water-cured concrete.

Table 2 Summary of Impact Loading Test Results

Test * Series		Secant Modulus in kg/cm^2	Initial Tangent Modulus in kg/cm^2	Time Duration of Loading in sec.	Comp. Strain Corresponding to Breaking Stress in %	Maximum Strain in %	Comp. Strength in kg/cm^2	Average Rate of Loading in $kg/cm^2.sec.$	Average Rate of Strain in 1/sec.
A	As	2.26×10^5	2.99×10^5	40.0	0.310	0.392	185	6.21	62.3×10^{-6}
	Ad	2.28×10^5	3.44×10^5	1.09	0.247	0.247	200	308	1606×10^{-6}
	Ad/As	1.26	1.15	----	0.56	0.63	1.08	----	-------
B	Bs	2.51×10^5	3.17×10^5	32.2	0.250	0.382	229	7.15	50.5×10^{-6}
	Bd	2.77×10^5	3.39×10^5	0.61	0.200	0.335	243	4.8	2490×10^{-6}
	Bd/Bs	1.10	1.07	----	0.80	0.88	1.06	----	-------
C	Cs	2.68×10^5	3.39×10^5	23.8	0.220	0.309	248	11.3	60.6×10^{-6}
	Cd	2.80×10^5	3.34×10^5	1.38	0.178	0.283	254	201	1334×10^{-6}
	Cd/Cs	1.04	0.985	----	0.81	0.92	1.02	----	-------
D	Ds	2.54×10^5	2.86×10^5	26.2	0.232	0.321	262	11.0	68.5×10^{-6}
	Dd	2.52×10^5	2.97×10^5	1.00	0.220	0.315	267	444	1818×10^{-6}
	Dd/Ds	0.99	1.04	----	0.95	0.98	1.02	----	--------
E	Es	2.85×10^5	3.77×10^5	25.4	0.172	0.242	329	11.2	56.5×10^{-6}
	Ed	3.04×10^5	3.66×10^5	0.94	0.180	0.227	312	494	1812×10^{-6}
	Ed/Es	1.07	0.97	----	1.05	0.94	0.95	----	-------
F	Fs	2.93×10^5	3.36×10^5	38.2	0.193	0.257	314	8.75	37.5×10^{-6}
	Fd	3.15×10^5	3.14×10^5	0.92	0.180	0.243	300	309	1476×10^{-6}
	Fd/Fs	1.07	0.94	----	0.93	0.95	0.96	----	-------

* Suffix " d " = Impact loading test, Suffix " s " = Static test.

* ACCURACY AND SIMPLIFICATION OF THE SEISMIC DESIGN *

BY SP. MENAYAS, CIVIL ENGINEER

1.- From an examination of the seismic problem, it becomes imperative to express the seismic coefficient not as a constant for all structures of the same geographical region, but as a function of:

a) The rigidity of the structure and the mass distribution, expressed in terms of the natural period T_s of the fundamental motion of the structure.

b) The physical property of the material of the structure and the ground.

On the other hand, since during the oscillations, the seismic coefficient of the structure changes as a function of time, the most unfavourable moment of the fundamental motion, which produces the maximum base shear, must be determined, taking into account the increase due to resonance and the decrease due to damping.

For these it is proposed to use the following terms:

a) External seismic coefficient C_o or seismic coefficient of the ground:

We define thus, the ratio of the observed maximum horizontal component of the seismic acceleration of the ground to the acceleration of gravity.

b) Mean or particular seismic coefficient C of the structure:

We define thus, the ratio of the maximum resultant of the lateral seismic forces (base shear) to the weight of the whole structure.
Hence, by the definition :

$$R = cW \cdots\cdots\cdots (1)$$

where: R = the resultant of the horizontal (lateral) seismic forces.

W = the weight of the structure.

Obviously, because all the points of the structure have not the same seismic acceleration, the centre of the seismic forces does not coincide with the centre of gravity of the structure.
The particular seismic coefficient C of every structure depends on its rigidity.
The measure of rigidity is given by the natural period T_s of the structure.
The maximum particular seismic coefficient C of the building, after an impulse of the ground motion, is produced in the case of stereostatic (perfectly rigid) structure and it is equal to C_o
The particular seismic coefficient of a structure can be written:

$$C = \lambda \mu C_o \cdots\cdots\cdots (2)$$

where: λ = a function of the rigidity

μ = a factor depending on the resonance and the damping.

2.- Calculation of the mean or particular seismic coefficient of a structure.

A.- The capacity of absorption of seismic kinetic energy by the structure is limited and depends on its rigidity.

By equating the work done by the ideal stereostatic structure and the real structure (elastic) during an earthquake impulse (half a ground oscillation), we have the relation :

$$C = C_o \frac{\sum_{k=1}^{n} a_k W_k}{a_o W} \quad \cdots\cdots\cdots (3)$$

where: W_k = the weight of the k^{th} floor

$$W = \sum_{k=1}^{n} W_k$$

n = the number of floors

a_o = the amplitude of the horizontal component of the motion of the ground

a_k = the translation of the k^{th} floor during the time of displacement a_o on the ground, Before the structure starts to give out the absorbed seismic energy (before the beginning of the vibration of the structure) (Fig.1).

Clearly the ratio :

$$\frac{\sum_{k=1}^{n} a_k W_k}{a_o W} = \lambda \quad \cdots\cdots\cdots (4)$$

is less than unity and is called rigidity factor.

When the velocity v of transmission of the motion of the ground in the overlying structure is given by the equation :

$$v = \frac{2H}{T_s} \quad \cdots\cdots\cdots (5)$$

where: H = is the height of the structure, the magnitude of the translation a_k is given by the equation :

$$a_k = \frac{1}{2} a_o \left[1 + \sin \frac{\pi}{2}\left(1 - \frac{2h_k}{H_o}\right)\right] \quad \cdots\cdots\cdots (6)$$

where: h_k = the distance from the foundation to the point A_k under consideration (Fig.1)

H_o = the distance from the foundation to the point M (Fig.1) over which the motion has not yet been transmitted when the foundation has moved through the amplitude $2a_o$

Hence:

$$H_o = v\frac{T_e}{2} = H\frac{T_e}{T_s} \quad \ldots\ldots\ldots (7)$$

where: T_e = the period of the ground forcing motion (earthquake)

From equations (6) and (7) we have:

$$a_k = \frac{1}{2}a_o\left[1+\sin\frac{\pi}{2}\left(1-\frac{2h_k}{H}\frac{T_s}{T_e}\right)\right] \quad \ldots\ldots\ldots (8)$$

For structures with storeys of constant height and constant concentrated static loads i.e. when:

$$H = n\,h_o \quad \ldots\ldots\ldots (9)$$

$$W = n\,W_k \quad \ldots\ldots\ldots (10)$$

where: n = number of storeys
h_o = the height of every storey,

the value of the rigidity factor λ is derived from equations (4), (8), (9) and (10) :

$$\lambda_p = \frac{\sum_{k=1}^{n} a_k}{n\,a_o} = \frac{1}{2n}\left[n+\sin\frac{\pi}{2}\left(1-\frac{2T_s}{nT_e}\right)+\sin\frac{\pi}{2}\left(1-\frac{4T_s}{nT_e}\right)+\right.$$
$$\left.+\ldots\ldots\ldots+\sin\frac{\pi}{2}\left(1-2\frac{T_s}{T_e}\right)\right] \quad \ldots\ldots\ldots (11)$$

For continuous uniformly distributed static loads, equation (11) becomes:

$$\lambda_p = \frac{\int_o^H a_k\,dh}{H a_o} = \frac{1}{2H}\int_o^H\left[1+\sin\frac{\pi}{2}\left(1-\frac{2h_k}{H}\frac{T_s}{T_e}\right)\right]dh \quad \ldots (12)$$

From equations (11) and (12) the tables I and II are set out, the first for concentrated masses on the floors (point loads) of the multistory buildings and the second for continuous mass distribution, along the height of the structure.

It is of special importance to note that the part A_oM and especially the region M (Fig.1) is subjected to the maximum danger of damage because of the small radius of curvature of the deflected structure.

The point M lies on the structure when it is slender and flexible.

Generally the total lateral seismic force R for one earthquake impulse is given by :

$$R=(\lambda_p W_p+\lambda_p W_p)c_o \quad \ldots\ldots\ldots (13)$$

where: λ_p = the rigidity factor for point loads,
λ_p = the rigidity factor for continuous loads,
W_p = the resultant of the point loads,
W_p = the resultant of the continuous loads.

B.- Resonance and damping

Definitions:

T_o = the duration of maximum seismic intensity. This is a function of the magnitude of the earthquake and can be taken equal to 6 seconds.
ρ = damping coefficient during half a period $\frac{T_s}{2}$ of the structure fundamental motion.

$$\rho=e^{-\frac{K}{T_s}} \quad \ldots\ldots\ldots (14)$$

where: K = Constant depending on the physical properties of the material. For reinforced concrete structures K can be taken equal to 0,20.

μ = Resonance factor i.e. factor of increase of the particular seismic coefficient of the structure because of resonance, taking into account the damping.
a_k = the amplitude of undamped oscillation of the point A_k (Fig.2) without the influence of resonance.
a'_k = the amplitude of oscillation of the point A_k after m ground impulses taking into account the effects of resonance and damping

Hence, by definition :

$$a'_k=\mu a_k \quad \ldots\ldots\ldots (15)$$

The value of the resonance factor μ can be calculated from the equation:

$$\mu=1+\rho\frac{\sin\left(\frac{\pi}{2}\frac{T_s}{T_e}\right)}{\sin\frac{\pi}{2}}+\rho^2\frac{\sin\left(3\frac{\pi}{2}\frac{T_s}{T_e}\right)}{\sin\frac{\pi}{2}}+\ldots\ldots+\rho^{m-1}\frac{\sin\left[(2m-3)\frac{\pi}{2}\frac{T_s}{T_e}\right]}{\sin\left[(2m-3)\frac{\pi}{2}\right]}+$$
$$+\rho^m\frac{\sin\left[(2m-1)\frac{\pi}{2}\frac{T_s}{T_e}\right]}{\sin\left[(2m-1)\frac{\pi}{2}\right]} \quad \ldots\ldots\ldots (16)$$

where: m = number of impulses of ground motion given by the relation:

$$m=2\frac{T_o}{T_e} \quad \ldots\ldots\ldots (17)$$

From equation (16) the table III is set out, for $T_e = 1,0^{sec}$ (soft ground).
If in equation (16) we introduce:

$T_e = 0,3^{sec}$ (for hard ground)
or
$T_e = 0,6^{sec}$ (for medium stiff ground),

we can set out two tables which with table III cover all needs of the aseismic design.

3.- Distribution of the total lateral seismic force along the height of the structure.

By shear deflection (Fig.3) of multistory structures the distribution of the total lateral shear R, given by the relation :

$$R = \sum_{1}^{n} P_k = cW = \lambda \mu c_o W \quad \ldots\ldots \ldots (18)$$

is obtained by the equation :

$$P_k = \frac{cW}{\sum_{k=1}^{n} (W_k \sqrt{h_k})} W_k \sqrt{h_k} \quad \ldots\ldots\ldots (19)$$

which assumes a paraboliodeflection of the structure.

When the deflection is due to shear and to bending (stacks) the distribution of the total lateral force along the height of the structure, can be obtained by the equation :

$$P_k = \frac{cW}{\sum_{k=1}^{n} (W_k h_k)} W_k h_k \quad \ldots\ldots\ldots (20)$$

which assumes a linear deflection of the structure. (Fig.4)

The lateral shear Q on the Kth floor (Fig. 3 and 4) is given by the relation:

$$Q_k = \sum_{k}^{n} P \quad \ldots\ldots\ldots (21)$$

4.- Distribution of the shear Q on the columns and walls.

For the distribution of the shear Q on the seismic load bearing vertical members, it is required to determine its point of application. Because the coordinates of the point of application of the shear Q are functions of the coordinates of the pole of the horizontal rotational oscillations of the slab-discs of the floor under consideration, whose position depends on the position of the centre of the seismic forces, the coordinates of the pole is determined by the method of successive approximations as a function of the coordinates of the centre of seismic forces by the following equations which are valid in the case that Q_x is parallel to the axis of Ox (Fig.5) :

$$y_{cx} = \frac{\varphi_x \sum y_k D_{kx} - Q_x}{\varphi_x \sum D_{kx}} \quad \ldots\ldots\ldots (22)$$

$$\varphi_x = \frac{Q_x (y_E - y_I)}{(y_E - y_I)(\sum y_k D_{kx} - y_{cx} \sum D_{kx}) + \sum x_k^2 D_{ky} - x_I \sum x_k D_{ky} + \sum D_{\omega k}} \quad \ldots\ldots (23)$$

$$y_E = \frac{j_x - y_{cx} S_x}{S_x - y_{cx} W_x} \quad \ldots\ldots\ldots (24)$$

$$y_I = \frac{\sum y_k D_{kx}}{\sum D_{kx}} \quad \ldots\ldots\ldots (25)$$

$$x_I = x_{cx} = \frac{\sum x_k D_{ky}}{\sum D_{ky}} \quad \ldots\ldots (26)$$

where: D_{kx}, D_{ky} = the indices of resistance in bending and shear of the columns and walls correspondingly in the direction of axes Ox and Oy

x_k, y_k = the coordinates of the centre of section of the column or wall k

φ_x = the angle of maximum horizontal rotational oscillation

x_E, y_E = the coordinates of the centre E of shear Q (seismic forces)

x_I, y_I = the coordinates of the elastic centre i.e., the centre of gravity of the indices D_{ky}, D_{kx}

$D_{\omega k}$ = the indices of resistance in torsion.

By the first step of calculation we assume that the centre E (Fig.5) of seismic forces, coincide with the centre of gravity G

The distribution of the resultant Q in the columns and walls is carried out by the following relation:

$$P_{kx} = \varphi_x (y_k - y_{cx}) D_{kx} \quad \ldots\ldots\ldots (27)$$

$$P_{ky} = -\varphi_x (x_k - x_{cx}) D_{ky} \quad \ldots\ldots (28)$$

where: P_{kx}, P_{ky} = the components of the seismicforce per column or wall correspondingly parallel to the axes

j_x, j_y = the moment of inertia of the static loads parallel to the axes

S_x, S_y = the static moment of static loads parallel to the axes.

W_x = the static loads.

Obviously, if the pole of rotational oscillations is taken as origin, the equation (24) becomes

$$y_E = \frac{j_x}{S_x} \quad \ldots\ldots (29)$$

Omitting the influence of the index of torsional resistance, the ordinate

Y_{Cx} of the pole of rotation is given by :

$$Y_{Cx} = \frac{\sum y_k^2 D_{kx} - Y_E \sum y_k D_{kx}}{\sum y_k D_{kx} - Y_E \sum D_{kx}} \quad \ldots\ldots\ldots (30)$$

5.- Indices of resistance and static calculation.

It is aimed to obtain the indices of resistance and the static calculation of a multistory frame by breaking it up into equivalent one-column multistory frames.

The indices of resistance D are calculated from the equations :

$$D = \frac{1}{\delta} \quad \ldots\ldots\ldots (31)$$

$$\delta = \left(\frac{\gamma h^3}{3J} + \frac{3h}{F}\right)\frac{1}{E} + \beta h \quad \ldots\ldots\ldots (32)$$

where: γ = coefficient of relative rotation of the tangents at the upper and lower ends of the columns or walls

h = the height of the column or wall

J = the moment of inertia of the section

F = the area of the section

β = the angle of elastic rotation of the tangent at the lower end (Fig.6).

The value of γ varies between 0,25, when the tangents remain parallel, after the displacements, and 1,0 in the case of free upper end.

The coefficient γ of the column AB is calculated by the relations :

$$\gamma = \frac{3\alpha - 1}{2} \quad \ldots\ldots\ldots (33)$$

$$\alpha = \frac{\Phi_{AB}^{B}}{\Phi_{AB}^{A} + \Phi_{AB}^{B}} \quad \ldots\ldots\ldots (34)$$

$$\Phi_{AB}^{B} = \frac{k_B^r + k_B^l + k_B^u}{k_B^r + k_B^l + k_B^u + k_{AB}} \quad \ldots\ldots (35)$$

$$\Phi_{AB}^{A} = \frac{k_A^r + k_A^l + k_A^o}{k_A^r + k_A^l + k_A^o + k_{AB}} \quad \ldots\ldots (36)$$

where: k_B, k_A = the stiffness value of the right (r), left (l), lower (u) and upper (o) members of the nodes B and A (Fig.6)

Φ^B_{AB}, Φ^A_{AB} = the degrees of fixity of the column or wall AB, correspondingly of the lower and upper ends B and A.-

The values of the degree of fixity vary from unity (complete fixity) to zero (free end).

For the calculation of degree of fixity Φ the lengths l' of the equivalent spans are introduced.

In the case of beams with undisplaced supports the equivalent span is given by the relation :

$$l' = l\left(1 - \frac{\Phi}{4}\right) \quad \ldots\ldots\ldots (37)$$

and the term stiffness value :

$$k = \frac{J}{l'} \quad \ldots\ldots\ldots (38)$$

In the case of free horizontal displacements of the ends of the columns the stiffness values become :

a) In the case of loading of the storey AB of the column by a horizontal force applied at the upper end A, the stiffness value of AB is given by the relation : (Fig 6)

$$k_{AB} = \frac{J}{1{,}5\, h_{AB}}, \quad \text{i.e.} \quad l' = 1{,}5\, h_{AB} \quad \ldots\ldots\ldots (39)$$

b) In the case of loading of the storey AB of the column by a moment M constant along its height AB (Fig.7), the stiffness value AB of column AB is given by the relation :

$$k_{AB} = \frac{J}{3 h_{AB}} \qquad \text{i.e.} \quad l' = 3\, h_{AB} \quad \ldots\ldots\ldots (40)$$

c) In the case of loading of the floor AB of the column, by a moment M constant along the height AB (Fig.7), the stiffness value of BC of the underlying floor BC of the column is given by the relation:

$$k_{BC} = \frac{\Phi^o_c J_{BC}}{3 h_{BC}}, \qquad \text{i.e.} \quad l' = \frac{3 h_{BC}}{\Phi^o_c} \quad \ldots\ldots\ldots (41)$$

where: Φ^o_c = the degree of fixity of the column, (floor BB), at the lower end C.

For the complete separation of a multistory frame-column from the multistory frame building and to take into account the transmitted actions from the remaining building frame, the spans of the beams are replaced by the following equivalent freely supported spans L (Fig.8) :

$$L_A = \frac{\varphi^u_A l'_A + \varphi^u_B l''_B}{\varphi^u_A + \varphi^u_B} \quad \ldots\ldots\ldots (42)$$

where: φ_A^u, φ_B^u = distribution factors of the moments.

The above determination of indices of resistance D and correspondingly the static calculation on the assumption of free horizontal displacement i.e., without developpment of horizontal reactions at the columns by the slab disc, is valid when the sections of the columns or walls remain the same or change proportionally from floor to floor, for all the colums or walls. In a different case the calculation is corrected by the introduction of the term "degree of prevention" ζ. (Fig.9)

$$\zeta = \frac{\Delta_{BC}}{\Delta_{AB}} = \frac{D_{BC}/\Sigma D_{BC}}{D_{AB}/\Sigma D_{AB}} \quad \ldots\ldots\ldots\ldots (43)$$

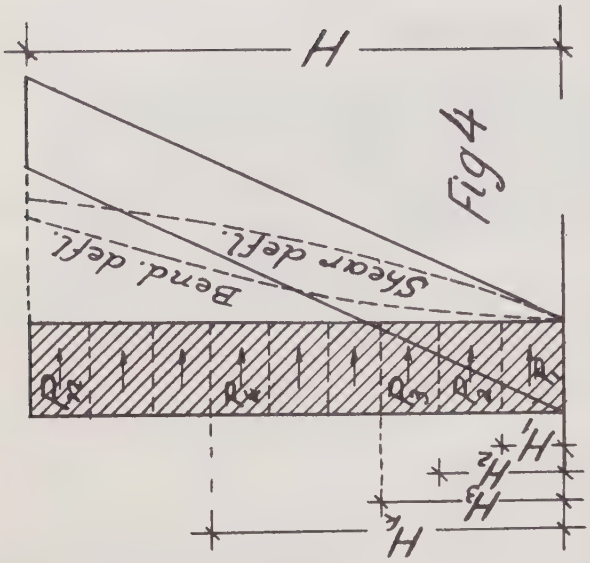
by means of the relation :

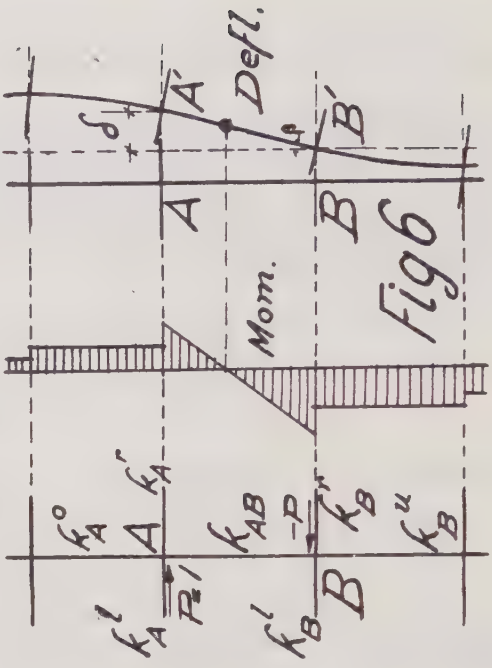
$$M_C^o = -M_B^u \frac{\Phi_c^o}{2} + \left(1 + \frac{\Phi_c^o}{2}\right) M_B^u \zeta$$

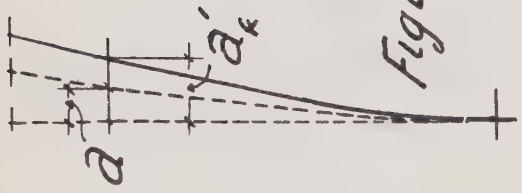
For the calculation of the degree of prevention we may accept with sufficient accuracy the indices of resistance as calculated by any approximate formula.-

Athens, March 1960.

SP. MENAYAS


H
Shear defl.
Bend defl.
Fig4


Defl.
A'
B'
A
B
Mom.
Fig6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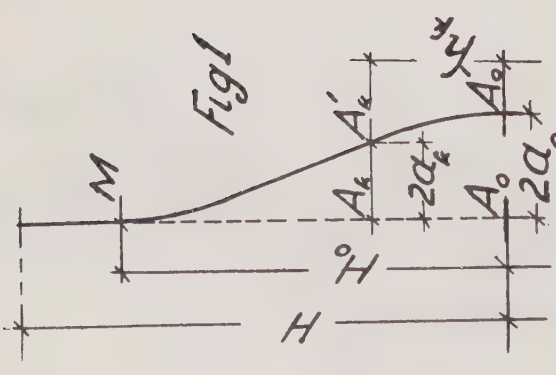
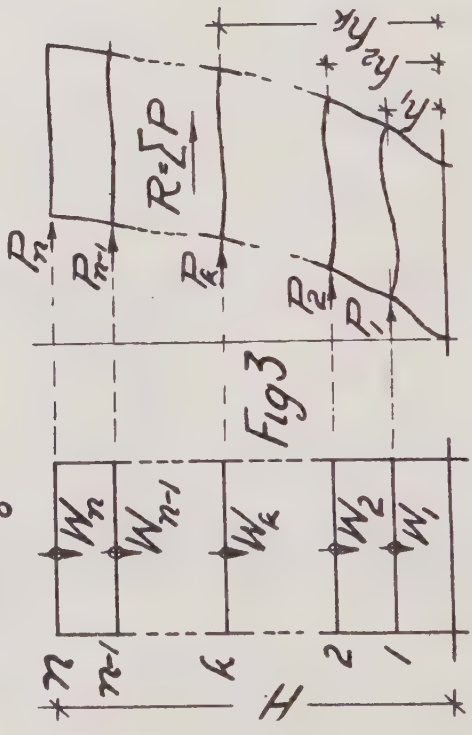


Fig1
H


R=ΣP
Fig3
H

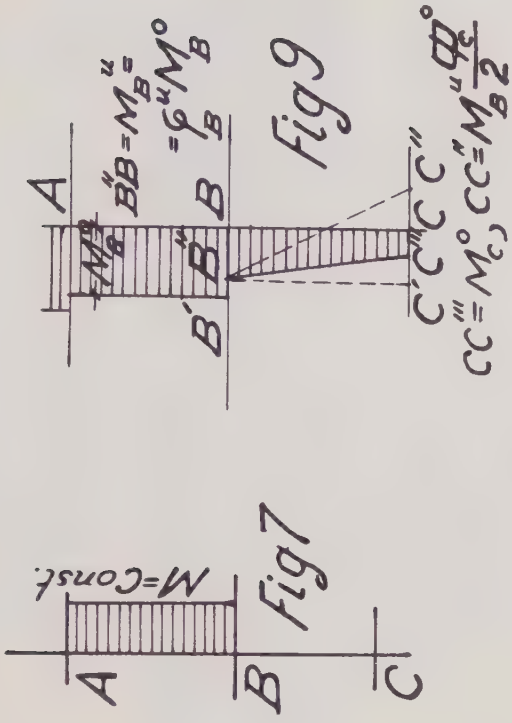


Fig7

Fig9

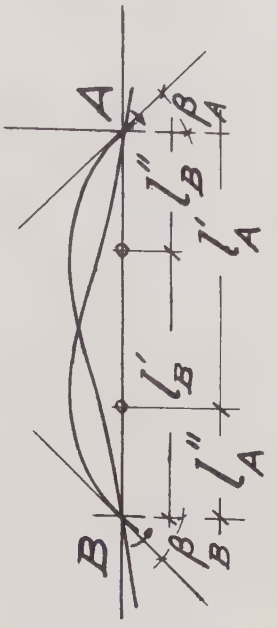


Fig8

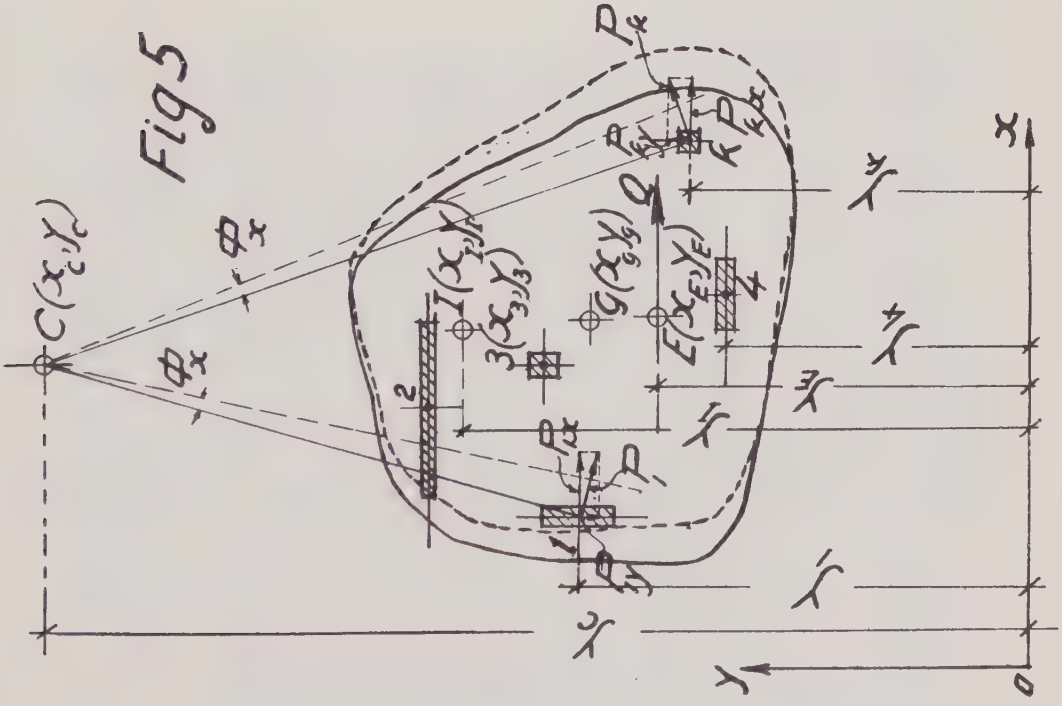


Fig5

Table I

Values of rigidity factor λ_p for equal concentrated static loads and storeys of constant height.

$\frac{T_s}{T_e}$	λ_p										$=n$
	1	2	3	4	5	6	7	8	9	10	
0,1	0,95	0,96	0,97	0,98							
0,2	0,82	0,89	0,91	0,92	0,92	0,93					
0,3	0,63	0,77	0,79	0,82	0,83	0,84	0,84	0,85	0,85		
0,4	0,43	0,63	0,67	0,71	0,73	0,74	0,75	075	0,76	0,76	
0,5	0,25	0,49	0,56	0,60	0,62	0,63	0,64	0,65	0,65	0,66	
0,6	0,12	0,38	0,46	0,50	0,52	0,53	0,54	0,55	0,56	0,57	
0,7		0,29	0,38	0,42	0,44	0,45	0,46	0,47	0,48	0,49	
0,8		0,22	0,31	0,35	0,37	0,39	0,40	0,41	0,42	0,43	
0,9		0,17	0,25	0,29	0,32	0,34	0,36	0,37	0,38	0,39	
1,0		0,13	0,21	0,25	0,28	0,30	0,31	0,32	0,33	0,34	
1,2			0,15	0,20	0,25	0,26	0,26	0,27	0,27	0,27	
1,4				0,15	0,21	0,22	0,22	0,23	0,23	0,23	
1,6					0,17	0,18	0,18	0,19	0,19	0,19	
1,8					0,11	0,12	0,13	0,14	0,15	0,16	
2,0						0,11	0,12	0,13	0,14	0,14	

T_s = natural period of the structure
T_e = period of ground motion
n = number of storeys

Table II

Values of rigidity factor λ_p for continuous loads
T_s = natural period of the structure
T_e = period of the ground motion

T_s/T_e	λ_p	T_s/T_e	λ_p	T_s/T_e	λ_p	T_s/T_e	λ_p	T_s/T_e	λ_p
0,1	0,95	0,6	0,60	1,1	0,34	1,6	0,23	2,5	0,15
0,2	0,92	0,7	0,52	1,2	0,31	1,7	0,22	3,0	0,12
0,3	0,85	0,8	0,46	1,3	0,29	1,8	0,21	3,5	0,11
0,4	0,77	0,9	0,41	1,4	0,27	1,9	0,20	4,0	0,10
0,5	0,68	1,0	0,37	1,5	0,25	2,0	0,19		

Table III

Values of resonance factor μ
Period of the ground motion $T_e = 1,0$ sec.
Duration of maximum seismic intensity $T_o = 6$ sec.
Number of impulses $m = 2\frac{T_o}{T_e} = 12$

T_s	μ	T_s	μ	T_s	μ	T_s	μ	T_s	μ
91	1,02	0,5	1,47	0,9	2,72	1,3	1,88	1,7	1,40
92	1,11	0,6	1,58	1,0	5,05	1,4	1,70	1,8	1,28
93	1,23	0,7	1,76	1,1	2,97	1,5	1,62	1,9	1,14
0,4	1,35	0,8	2,10	1,2	2,23	1,6	1,52	2,0	1,00

T_s = natural period of the structure
T_e = period of the ground motion
$[T_s]$ = sec.

Sp. Menayas- Civil Eng.

Supplement to the paper under the title

" accuracy and simplification of the seismic design

Equation (3) and (4) of the above mentioned paper, must be substituted by the following relation (3a) which is obtained from the ratio of the equations which express respectively the total lateral seismic force of the elastic structure and that of the stereostatic (rigid) structure

$$\lambda = \frac{C}{C_o} = \frac{\sum_{K=1}^{n} a_K W_K}{a_o W} \quad \ldots\ldots\ldots (3a)$$

Relations (11) and (12) are respectively substituted by the following equations (11a) and (12a).

$$\lambda_p = \frac{\sum_{K=1}^{n} a_K}{n a_o} = \frac{1}{2n}\left[n + \sin\frac{\pi}{2}\left(1 - \frac{2T_s}{nT_e}\right) + \right.$$

$$\left. + \sin\frac{\pi}{2}\left(1 - \frac{4T_s}{nT_e}\right) + \ldots\ldots + \sin\frac{\pi}{2}\left(1 - 2\frac{T_s}{T_e}\right)\right] \ldots\ldots (11a)$$

$$\lambda_p = \frac{\int_o^H a_K\, dh}{H a_o} = \frac{1}{2H}\int_o^H \left[1 + \sin\frac{\pi}{2}\left(1 - \frac{2h_K}{H}\frac{T_s}{T_e}\right)\right] dh \ldots\ldots (12a)$$

Athens, April 1960.

ASEISMIC PROVISIONS FOR THE FEDERAL DISTRICT, MEXICO

By

Emilio Rosenblueth*

Introduction

Before the earthquake of 28 July 1957 Mexican engineers knew that the Federal District Building Code was obsolete, that it was systematically violated, and that there was more need for research on structural responses to earthquakes. (1) But no one realized the tragic consequences that the inadequacy and ineffectiveness of the code and our ignorance in matters of aseismic design were to bring about.

Despite the aftermath, a true perspective must deem the lesson learned at a low cost indeed. Reaction was immediate; authorities took effective steps; a unique opportunity arose for gaining experience and even some insight into structural behavior; and the event attracted some of the best engineers from other parts of the world, foremost among whom were R.W. Binder, G.W. Housner, and the late Murray Erick, who awakened consciousness of the pertinent variables in aseismic design.

Emergency Regulations

One week after 28 July, the Building Department appointed a committee to write an emergency code. Regulations were drafted in three weeks and immediately enforced. They were later revised. Essentially unchanged, they were enacted in a presidential decree 12 September 1957. The chapter on aseismic design of the code extant at the time was abrogated.

The 1942 code (1) called for base shear coefficients of between zero and 0.10 depending only on the importance of the building, with 33 percent increase in allowable stresses. Office and apartment buildings should be designed for 0.025 g. Acceleration should be assumed uniform throughout the building. Structures up to 16 m (52.5 ft) tall needed not be designed against earthquake forces. The code included no provisions on torsion, none on masonry walls and partitions, and did not leave room for dynamic analysis. It was often applied in conjunction with the portal method of analysis with no regard for relative rigidities.

The earthquake brought to the foreground several points: (2,3,4)

1. Motions with recurrence period of 50 years caused far greater accelerations than the code called for, specially in the upper stories.

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2. The Valley of Mexico exhibited prevailing ground periods. Flexible structures on soft soil and rigid structures on hard ground were more vulnerable. This was particularly true of the former, because of the prevalence of distant epicenters and hence long periods.

3. Due regard should be given the question of relative rigidities, their relation with torsion and the contribution of masonry walls and partitions.

4. Great precision could not be expected in calculation of rigidities and torsional effects, particularly when masonry walls and partitions act with the rest of the structure.

The emergency regulations reflected these points. Aseismic design was required for all except the most unimportant buildings, independently of their height. Base shear coefficients were stepped up to between 0.05 and 0.10 for ordinary buildings, depending on the type of structure and its location in the Valley of Mexico (whether on the lake, transition, or hilly zone), and doubling these values for buildings of exceptional importance.

A linear variation of horizontal accelerations was adopted, from zero at the base of the structure to a maximum at the roof. Parapet walls and appurtenances were assigned an acceleration of 0.5 g. Setbacks received no special consideration.

It was specified that the computed eccentricity in torsion be increased by an "accidental" value, equal to $\pm$ 0.05 of the floor dimension measured at right angles to the acceleration. This applied to all buildings except warehouses, for which the coefficient 0.05 was increased up to 0.07 to cover unfavorable distributions of load. Besides, in no case should the design eccentricity be less than half of the maximum found in stories below the one under consideration, nor the torque be less than half of the maximum found in all stories above the one being analyzed.

It was felt that the prevalence of long periods in the lake zone precluded reductions in overturning moments. No provision was left in to allow such reductions. Shortly it became apparent that design complying with the regulations resulted in very costly structures. A round-table discussion was held at the Building Department with some consulting engineers. Reduction coefficients of up to 40 percent were adopted.

Thought was given to the possibility of requiring that all masonry be reinforced. Economic considerations discarded this, since unreinforced masonry costs one tenth of what it does in the U. S. while reinforcing steel can be obtained for about the same price as in that country.

Unreinforced masonry was to continue in general use. It was clear from the 1957 earthquake that such walls were prone to break leaving

the rest of the structure unaided to resist further motion, be it due to wind, the some quake, or aftershocks. A check on the conditions of the unaided frame was felt mandatory. Accordingly, the regulations required two analyses: one taking in consideration unreinforced walls, and the second assuming that these elements had failed and using a lower base shear coefficient.

In conjunction with conventional design (essentially 1940 American practice) the regulations allowed doubling most working stresses under combined static seismic loads. For concrete in diagonal tension, only a 33 percent increase was permitted; structural grade reinforcement could be designed at 90 percent of its yield point; and other grades at 80 percent of their nominal limits of elasticity. The regulations also permitted plastic analysis and design with a load factor of 1.5 in flexure and 1.2 in axial compression.

It was felt that with high base shear coefficients and high working stresses a well balanced design would be attained. For example, points of counterflexure shifted thus over a wider range than with the conventional small lateral loads and 33 percent overstress. Also, cross bracing did not turn out too flimsy in relation to the columns. (2)

The 33 percent overstress was retained for diagonal tension in concrete since the revision was to cover no more than aseismic design and it was realized that concrete members were being underdesigned in shear.

Drift was limited to 0.002 of story height except for lightweight steel roofs in factory constructions, for which no limitation was stated.

Buildings over 45 m (140 ft) tall required dynamic analysis, the details of which the regulations left unspecified.

The 1959 Revision

Early in 1959 the Building Department reorganized its Committee on the Building Code. It was the general consensus that the code merited thorough revision rather than mere polishing. Several subcommittees were appointed to study specialized topics, including one on aseismic design.

Most engineers who had applied the emergency regulations felt that ordinary buildings were being penalized in excess because some vulnerable atypical structures had been subjected to very high accelerations in 1957. Many who had decried Fate, resonance, and the earthquake's violence as responsible for the havoc had an opportunity to take a close look at the damaged structures when repairing them. In most ordinary buildings they had to ascribe damage to poor construction practices and oversimplification in analysis and design. High seismic coefficients were justified for atypical structures such as those resting on a single row of columns, but design of ordinary buildings could be liberalized, provided due attention was given the questions of relative rigidities,

of pounding with neighboring buildings, and of apparently minor structural details. This situation in aseismic design was probably decisive in leading to revision of the entire code.

There follows an annotated translation of the proposal made by the subcommittee, as it stood in December 1959. It may suffer minor changes before final approval.

1. Nomenclature. Each symbol is defined where it first appears. The most important are:

a = coefficient used in dynamic analysis (dimensionless);
c = V/W = base shear coefficient (dimensionless);
F_i = horizontal force at floor i (ton);
f'_m = least of brick or mortar compressive strengths, determined from standard tests (kg/cm^2);
H = elevation above base of structure (cm);
h = story height of wall panel (cm);
L = length of wall panel (cm);
n = number of stories of building considered (dimentsionless);
T = natural period (sec);
V = horizontal shear at base of building (ton);
W = weight of building (ton);
W_i = weight that is assumed concentrated at floor i (ton);
x = maximum computed horizontal displacement at height H (cm).

2. Zoning. According to subsoil characteristies the Federal District is divided in two zones conforming to the chapter on foundations.

The transition zone was incorporated in the lake zone together with all recent uncompacted fills in the hilly area. Despite statistical evidence favoring distinction between lake and transition zones, seismic intensity in other cities has been particularly violent near the slopes of hills and mountains. Conceivably a different earthquake from that of 1957 might make the transition zone its favorite prey. Moreover, parts of this zone were vacant of tall buildings to stand as witnesses of local intensity.

Local inexperience with loose fills suggests classifying them together with soft ground; they are so grouped in the chapter on foundations.

3. Classification of Buildings. Considering their intended use, constructions are classified in three groups:

Group A. Government, municipal, and public buildings (as pumping stations, powerhouses, telephone exchanges); those whose functions are especially important immediately following a seism (as hospitals); those containing articles of exceptional value (as museums); and those in which there is frequent conglomeration of people (as schools, stadiums, theaters, temples, terminal communication stations).

Group B. Buildings intended for private habitation or public use where conglomerations are uncommon: hotels, apartment and office buildings, industrial buildings, gas stations, restaurants, etc. Also fence walls with height exceeding 2.50 m, constructions housing costly goods or equipment, and those whose failure could endanger other buildings in this group or in group (A)

Group C. Isolated constructions not requiring the intervention of a registered engineer or architect and the failure of which, from earthquake effects, would not normally damage human beings, costly goods or equipment, or structures in the first two groups.

The following classification is based on structural characteristics.

Type 1. Structures having, at right angles with the direction analyzed, two or more elements capable of resisting horizontal shear, and whole deformations, under lateral loads in the direction in question are essentially due to flexure in structural members. It includes buildings having frames that consist of girders (or trusses or flat slabs) on steel or reinforced concrete columns, provided each frame is capable of resisting in all stories at least 50 percent of the design shear it would be called upon to resist if isolated from the rest of the structure, not requiring therefor the aid of walls or diagonal bracing. Floors and roofs must be sufficiently rigid and resistant to distribute the lateral forces among elements of various flexibilities.

Type 2. Structures whose deformations under lateral loads in the direction considered are due essentially to shearing stresses or axial forces in the structural members. Included are buildings supported by bearing walls only and those whose frames are incapable of resisting by themselves in each story 50 percent of the design shear. Each frame in this group must be able to resist at least 25 percent of the design shear.

Type 3. Elevated water tanks, chimney stacks, and all structures supported by a single column or row of columns perpendicularly to the direction analyzed, or whose columns are not tied at the roof and floors through elements of sufficient rigidity and strength to distribute the horizontal forces among columns of different flexibilities.

The need to differentiate groups (A) and (B) was made plain in 1957. The children's hospital and the country's largest hospital were severely damaged. Had the quake occurred at another time of day it would have been impossible to accomodate enough patients. The most conservative building code will not afford absolute protection against the most violent earthquake possible, nor is it clear that such an upper limit exists. (The most plausible bases for establishing an upper limit to the magnitude of earthquakes (5) are contradicted by the authors in their analysis of the Lisbon earthquake.) The fact that all authorities agree on seismic regionalization in terms of regional seismicity — not only of local

geology -- implies that they admit designing for magnitudes smaller than the maximum possible, even if that maximum is finite. Hence one must contemplate collapse, however remote. Clearly some buildings should be protected against collapse more dearly than others. Finally, in any probabilistic approach to structures, design loads must depend on the importance of the structure, especially the most unpredictable types of load.

The division into types classifies structures in a decreasing order of their ability to absorb energy. It is influenced by the code approved by the SEAOC in October 1958, (6) as well as on observations of damage inflicted by the 1957 quake. Differences with the SEAOC code reflect personal opinions, since neither system is based on quantitative information.

Experience with the emergency regulations indicates that the double check in stability computations -- with and without walls --, although desirable, was too demanding. The requirements that each frame of structures type 1 resist at least 50 percent of its own laternal load was left in as second best choice and within reach of most structural engineers. In part, also, it takes care of miscalculation of relative rigidities. The last comment applies also to the 25 percent minimum for structures of type 2.

4. Allowable Stresses. When using elastic methods of analysis and design for the combination of gravity and earthquake loads the following increases in allowable stresses are permitted with respect to those specified elsewhere in this code for gravity loads alone.

(a) In timber and structural or reinforcing steel, 50 percent.
(b) In other metals, concrete, and masonry, 33 percent.
(c) In the cohesion of soils, 100 percent.*
(d) In the angle of internal friction of soils, none.*

* Subject to revision by the subcommittee on foundations.

Some allowable stresses for static loads are as follows. Rolled structural grade steel in tension or compression, 1400 kg/cm^2 (20 ksi); other grades, 0.6 fy (fy at 0.2 percent offset). Structural grade bars as main reinforcement or stirrups in concrete, 1265 kg/cm^2 (18 ksi); other grades, 0.5 f_y. Concrete in compression due to flexure, 0.45 f'_c; in compression due to axial load, 0.28 f'_c; and in diagonal tension or shear, the average stress = $0.25\sqrt{f'_c}$ in kg/cm^2 ($0.94\sqrt{f'_c}$ in psi) plus a term depending on longitudinal reinforcement and shear span, but not to exceed $0.45\sqrt{f'_c}$ ($1.7\sqrt{f'_c}$ in psi).

Moduli of elasticity are 2.1×10^6 kg/cm^2 (30×10^6 psi) for steel, $15{,}000\sqrt{f'_c}$ in kg/cm^2 ($56{,}800\sqrt{f'_c}$ in psi) for concrete, and 250 f'_m for masonry. The last value is based on tests with materials from the Federal District. Moduli of rigidity for concrete and masonry are 0.4 of the moduli of elasticity. Use of empirically determined moduli in conjunction with standard strength of materials is expected to give good results for most cases on the basis of available test data. (7)

5. Base Shear Coefficients. The base shear coefficient in the ratio of the horizontal shear V at the base of the building to the weight W of the building. Live loads assumed in the evaluation of W are those tabulated in the chapter on loads.
In static analysis of buildings belonging to group (B), the following minimum values of C shall be used.

Type of structure	Zone of high compressibility	Zone of low compressibility
1	0.06	0.04
2	0.08	0.08
3	0.15	0.10

For buildings in group (A) these values shall be multiplied by 1.3. For those in group (C) aseismic design is not required.

The coefficient 0.06 was derived from the base shear registered in the 43-story Torre Latino Americana in 1957 (between 0.03 and 0.04 of its weight, (8)) somewhat increased to take into account the length of natural periods in this structure. The coefficient 0.15 responds to the fact that the two cases of resonance in 1957 indicated values greater than 0.10 for structures type 3. (4) The base shear coefficient of 0.10 for structures type 3 on soil of low compressibility is based on experience in California, allowing for lesser seismicity and higher working stresses in Mexico City. Other values in the Table were fit in once these had been agreed upon.

The 30 percent increase for group (A) is much smaller than the 100 percent in the emergency regulations and even smaller than the variations required by the 1942 code. The fact that such a small increase is now thought adequate comes from the feeling that the coefficients are quite high as they stand, plus experience gained in applying the emergency regulations, which led to the conclusion that greater variation would excessively increase the cost of structures. There was besides a qualitative consideration of the results of probability analyses concerning the maximum response of damped systems to earthquakes of given intensity and the statistical distribution of intensities in time.

6. Selection of Method of Analysis. The method of Art. 7 is acceptable when the following requirements are simultaneously met.

- In each story at least 75 percent of vertical loads is supported by walls joined through a continuous slab.

- There exist in the perimeter at least two bearing walls, parallel to each other or forming between them an angle not greater than 20 degrees. These are joined through the slab for a length at least equal to half that of the building measured along these walls.

- The ratio of height to width of the building does not exceed 1.5.

- The aspect ratio of the base of the building does not exceed 2.0 except if, for aseismic design, it can be assumed divided into independent segments with maximum aspect ratio within this limitation and each segment is earthquake resistant in accordance with Art. 8.

In all other cases recourse must be made to the static or the dynamic method, save that the static method is not applicable when in any story torsional eccentricity exceeds 20 percent of the plant dimension measured parallel to the eccentricity.

The need for a simplified method applicable to inexpensive construction arose mostly from the fact that the 1942 code did not require aseismic design of buildings up to four stories tall. Of the six total collapses of 1957, three were in this range as well as many cases of major damage; hence, the emergency regulations required aseismic design of practically all structures. Since many houses and four-story buildings are designed by registered practitioners with little academic background, there was need to simplify matters in this range. Something similar to the present article had been unofficially approved by the Building Department shortly after issuance of the emergency regulations.

Analysis of more than 100 buildings (each with 2-16 degrees of freedom but no torsion) by the methods of Arts. 8 and 9, disclosed not a single simple criterion to discern the need for dynamic analysis. In most cases the static method gave horizontal shears greater than their dynamic counterparts. The taller structures systematically gave dynamic shears considerably smaller than those computed statically. Only in systems having torsional eccentricity variable with height were important errors on the unsafe side found when applying the static method. Accordingly it was decided not to exact a dynamic analysis at all, but to leave the choice up to the structural engineer, except in cases of high torsional eccentricities.

7. Simplified Method of Analysis. Lateral displacements, torques, and overturning moments may be ignored. It suffices to check that the total shear acting in each story, computes as in Art. 8, does not exceed the sum of the projected shear strengths of the bearing walls on axes parallel to the directions in which the acceleration is assumed to act. The check shall be made along two mutually perpendicular directions.

In wall panels whose ratio of story height, h, to length L, exceeds 1.33, the allowable stress shall be reduced by a factor $(1.33L/h)^2$.

For purposes of this article the shear strength of bearing walls, already increased because it applies to lateral loading, shall be taken as 0.5 kg/cm^2 (7 psi) for walls of light-weight brick, and 1.5 kg/cm^2 (21 psi) for those of burned clay, unless if, by means of the tests specified in the chapter on masonry construction, the value of f'_m is determined. In that case the shear strength shall be taken equal to 0.06 f'_m allowing for no increase derived from vertical loads.

Walls in question shall comply with the chapter on masonry. It shall be unnecessary to satisfy Art. 15 in the present chapter.

The reduction factor tends to take estimatively in consideration the decrease in rigidity with increase in the h/L ratio.

8. Static Analysis. Design shears at different elevations in a building shall be computed from the assumption of linear distribution of horizontal accelerations, varying from zero at the base of the structure to a maximum at the top, in such a manner that the ratio V/W at the base equal the value of C specified in Art. 5. Accordingly, the horizontal force at floor i is given by the expression

$$F_i = CW \frac{W_i H_i}{W_1 H_1 + W_2 H_2 + \ldots + W_n H_n}$$

Tanks on buildings, constructions not forming an integral part of the structure, and all other elements whose stability depends on local acceleration (not on shear or overturning moment) shall be designed for an acceleration not smaller than twice the one obtained from the forgoing paragraph nor than 0.15 times gravity. The requirement applies to cantilever walls, signs, ornaments, windows, partitions, curtain walls, façade veneer, etc., and to horizontal diaphragms that transmit the inertia forces of the masses they carry.

For design purposes, overturning moments may be reduced, but for each frame or group of resisting elements it shall not be taken smaller than the design shear at the elevation considered times its distance to the center of gravity of all the corresponding masses above.

Torsional eccentricity must be computed in all stories. The design eccentricity shall be taken equal to 1.5 times the computed value* plus or minus 0.05 times the maximum plant dimension at the elevation considered (accidental eccentricity), measured at right angles with the shear (Fig. 1). For each structural member, that sign of the accidental eccentricity shall be taken which gives the most unfavorable condition. If uniformity of live loads and rigidities can be assured, the accidental eccentricity may be lowered slightly.

*Subject to revision in the light of further dynamic analyses before submission for final approval.

Provisions for appurtenances are less stringent than what other codes require. Besides, it was felt that local accelerations should be taken in consideration. The minimum of 0.15 g corresponds to type 3 in the table of Art. 5, for a parapet resting on the foundation can be regarded as a structure of this type.

The dynamic analysis of several buildings referred to above did not disclose the need to include special provisions for setbacks in buildings up to 16 stories tall. For much taller structures, dynamically computed shears would be smaller than those found in static analysis.

Explicit consideration of natural periods in this method was not deemed appropriate. Expirical formulas for the fundamental period can well err over 100 percent; and if this period is to be computed by trustworthy methods, a little more effort will permit application of the dynamic method.

Allowable reduction in overturning moment is conservative. It amounts to about 25 percent at the base of the building (near 30 percent in extreme cases), varying to zero at the uppermost story. The latter condition is imposed by equilibrium. This conservative stand in connection with overturning was based partly on the fact that the code's design criteria for columns and for distribution of soil pressure (Art. 19d) are somewhat liberal and partly on the predominance of distant-focus (hence long-period) earthquakes in the Federal District. (9)

Accidental torsion is specified much in the same manner as in the emergency regulations (an additional rather than a minimum eccentricity as in the SEAOC code). However, no distinction is made between warehouses and other buildings since live load distribution contributes far less to accidental eccentricity than chance variations in rigidity and errors in its computation.(9) The 1.5 factor is an approximation to Housner's dynamic magnification of torsion in single-story buildings.(10)

Emergency regulation reguirements for torsion in terms of that computed in stories other than the one considered are qualitatively correct. (9) They were dropped for the sake of simplicity, since dynamic analyses showed that these effects were quite small when the maximum eccentricity does not exceed 0.20 of the plant dimension.

9. Dynamic Analysis. Any dynamic method of analysis is permissible if it is consistent with the nature of earthquakes and behavior of ground and structure and fulfills the restrictions in this article.

If the structure is analyzed computing its natural modes of vibrations, it shall be assumed that the ground suddenly undergoes a constant acceleration equal to aC times gravity. Here C are the coefficients of Art. 5 and a the values obtained from the following expressions, in which T represents the natural period in seconds.

(a) In the zone of low compressibility, for $T \leq 0.5$, $a = 1.0$, and for $T \geq 0.5$, $a = 0.5/T$ (Fig. 2a).

(b) In the zone of high compressibility, if $T \leq 1.0$, $a = 0.5\ (1+T)$, if $1.0 \leq T \leq 2.5$, $a = 1.0$, and if $T \geq 2.5$, $a = 2.5/T$ (Fig. 2b).

With basis on quantitative experimental information a different relation between a and T may be adopted if approved by the Building Department.

When using this method design shall be based on the square root of the sum of squares of the generalized forces (shears, overturning moments, bending moments, etc.) corresponding to the various modes in the direction considered. Natural modes whose combined effect does not modify seismic design shears by more than 10 percent may be neglected. In this phase of analysis it is also permissible to neglect the dynamic effect of statically computed torques amounting to eccentricities not greater than 0.2 times the plant dimention measured parallel to the eccentricity. Effects of these as well as of the accidental eccentricity shall be taken in consideration in accordance with Art. 8.

At no elevation shall strength be provided for less than 60 percent of the horizontal design shear obtained from static analysis.

Requirements of the static method concerning walls, ornaments, signs, etc. shall be complied with in design, although it is not necessary to double accelerations computed dynamically.

The subcommittee is conscious of limitations in the method proposed. Yet, experience with the emergency regulations pointed the need for setting rules of dynamic analysis. The wording does not exclude the use of methods more rational than the one specified.

The curve giving a vs. T on firm ground is based on analyses of acceleration spectra, with 10 percent of critical damping, from earthquakes recorded in Western United States. (11) Results of the analyses have been modified due to the prevalence of earthquakes with distant focus. Except for the value of T at which the horizontal line intersects the hyperbola, the curve conincides with those in the San Francisco and Russian codes.

The shape of the curve in Fig. 2b intends to account for long prevailing periods and damping of short-period waves in the compressible mantles.

Combination of natural modes by the root mean square criterion is based on probability analyses.(12)

The limitation of 60 percent of the static-method shears is safer and just as simple as a cutoff in the base shear coefficient for very long natural periods.

The chance given to deal statically with small eccentricities seems artificial. It is necessary since dynamic analysis of buildings with three degrees of freedom per floor is far from simple, and it would not be justified to require explicit consideration of small torques when great accuracy is not expected in the determination of the center of torsion.

10. Drift Limitations. The maximum relative displacement between consecutive floors shall not exceed 0.002 times the corresponding difference in elevations, save where elements that do not form an integral part of the structure are attached thereto in a manner such that they are not damaged by its deformations. The value 0.002 may then be increased to 0.003 in the zone of high compressibility and to 0.004 in the rest of the Federal District; in floors and roofs that do not usually support live load, no limitation is imposed.

In computing horizontal displacements every element which forms an integral part of the structure shall be taken in consideration.

Drift limitations may seem overly conservative in the light of experience in other cities. Actually, on soft ground they are intended to minimize panic(9) and in all cases to minimize property damage, for to an extent it should be one of the code's purposes to protect the interests of those who buy a buildings, not only the lives of those who inhabit it.

11. Special Provisions in Façades. In interior and exterior facades, window panes shall be placed in their frames leaving a play equal to at least twice the relative horizontal displacement. This measure may be omitted when the window frames are so attached to the structure that its deformations can not affect them.

12. Awnings. Buildings taller than 10 m (33 ft) faced with materials that may fracture, or having glass mounted on frames fixed to the structure in a manner incapable of preventing damage from structural deformations, shall possess an awning cantilevering at least 1.5 m all along the facade. This protection is not required if the facade is set 1.5 m or more back of the property line and there is no open pass to pedestrians in this area.

13. Elements not Forming an Integral Part of the Structure. Regarded as such and as not contributing to structural strength and rigidity are those parts of a building which are made of brittle materials, such as glass and plaster, and partitions less than 10 cm (4 in.) thick.

Metal and wooden partitions and those made of highly deformable materials, such as glass-reinforced plastics, when not possessing panels of brittle materials, require no special provisions in their junction with the structure from the point of view of lateral displacements. All other nonstructural elements must be fixed to the structure taking such measures that they not be damaged by the latter's deformations. These precautions may consist in fixing the element at one floor, leaving it free at the opposite end, or placing it on one floor and guiding it by means of a channel at the opposite end, etc. Leeways shall be left consistent with the displacements of the structure, and stability verified in accordance with the accelerations of Art. 8.

14. Structural Elements. Elements in a building shall be regarded as structural when, being of metal, timber, concrete, or masonry, their parts are tied to each other and to the rest of the structure.

Masonry walls and partitions forming part of the structure must satisfy the following requirements.

(a) They shall be no thinner than 10 cm (4 in.).

(b) At least in one direction they shall possess or be tied to metal or reinforced concrete members (girders, bond beams, slabs, columns, pilasters, etc.) whose spacing does not exceed 25 times the wall thickness when $f'_m = 50$ kg/cm^2 (711 psi). If $f'_m = 50$ kg/cm^2 the spacing, in centimeters, shall not exceed $f'_m/2$, where f'_m is expressed in kg/cm^2. (This requirement may be omitted in reinforced masonry.)

(c) Blocks, brick, and tile shall have a compressive strength not smaller than 15 kg/cm^2 (213 psi).

(d) Mortar shall have a compressive strength not smaller than 30 kg/cm^2 (427 psi).

This and the following article are supplementary to the chapter on masonry. Therein specific rules are given for bond beams and pilasters, calling for a minimum of four longitudinal bars to give an area not smaller than $0.05\ f_y/f'_c$ times the cross section area of the member or times the square of the wall panels whose height to width ratio exceeds 1.5, around openings whose maximum dimensions is less than four times the wall thickness, and in reinforced masonry work.

When applying the simplified method of analysis compliance with the requirements in that chapter presumably makes it unnecessary to compute stresses in the wall-reinforcing elements.

15. Bond Beams and Pilasters. Reinforcement of bond beams and pilasters shall resist in tension the components of stresses in the wall or partition assuming cracks at 45 degrees with the horizontal. Stresses due to dead load may be taken into account. Bond of the reinforcement shall checked. Continuity shall be attained in the transmission of forces taken by the reinforcement. Provisions shall be made to resist the resultant of tension where bars are bent. Bars shall be anchored by means of standard hooks at free edges of panels.

16. Clear Spacing at Property Lines and in Construction Joints. Every new building shall be constructed away from neighboring pieces of land at least 3 cm (1.2 in.), but not less than $x + 0.006\ H$ in the zone of high compressibility nor less than $x + 0.004\ H$ elsewhere. Here x = maximum lateral displacement computed at elevation H. When using the simplified method of analysis take $x = 0.002\ H$.

The exterior surface of every bearing wall in the perimeter of the building shall be vertical.

At construction joints the same criterion applies as for property lines unless special measures are taken to prevent damage from pounding.

Emergency regulations called for "special studies" to prevent pounding. The requisite had practically nil effects in design practice. Hence the subcommittee decided to be more explicit.

The possibility of specifying the clear space as a function of the existence of neighboring structures was dismissed because it might cause legal difficulties. The spacing approved intends to account for computed seismic displacements plus uncomputed rocking and, to a degree, tilting due to differential settlement.

17. Repair of Damage Structures. When a building suffers minor earthquake damage either in its structure of finishing, it shall be repaired restoring at least its original resistnace. If the facades have been damage, the corresponding requirements in Arts. 8, 11, and 12 shall be fulfilled. If the damage is such as to endanger structural stability, the structure shall be reinforced complying with all articles in the present chapter which pertain to structural stability.

Repair of earthquake damage shall in all cases be preceded by a permit from the Building Department. If repairs have been made without it, the Building Department may condemn the repaired members so as to effect repairs according to an approved design.

Actually a permit is required for all repairs whatever the cause. It is appropriate to insist in connection with earthquake damage since it has been Mexico's experience that inadequate repairs run rampant immediately following a macroseism. It was found impracticable to be more explicit in this article.

18. Precautions during Construction. All work shall be so executed that, during the construction stage, the building be capable of resisting the lateral forces that correspond to the base shear coefficient specified in Art. 5. In secondary members it suffices to take measures preventing damage from local failure.

19. General Criteria of Stress Analysis. Simultaneous action of earthquake and wind need not be considered. Structural analysis of buildings subjected to the combined effects of gravity and earthquake shall satisfy at least the requirements specified below, with the exceptions pertinent to the simplified method.

(a) Effects of lateral forces shall be analyzed taking in consideration the horizontal displacements and rotations of all structural members, as well as their continuity and their rigidities.

(b) Flexural deformations of walls and those due to axial load in columns shall be taken in consideration whenever they appreciably affect design stresses and displacements.

(c) In steel structures embedded in reinforced concrete it is permissible to consider the combined section for computing stresses and rigidities if connectors are provided, capable of preventing slip between both materials, and the requirements in the chapter on compound structures are met.

(d) If analysis indicates tensions between foundation and ground, it shall be assumed that part of the foundation is exempt from contact pressures. Equilibrium of the total computed forces and moments must be satisfied in the rest of the contact area and the allowable stresses on the soil shall not be exceeded. Stability of the foundation and superstructure shall be checked under these conditions. If there are elements such as piles or caissons capable of taking tension, they shall be considered in the analysis.

(e) In all cases a revision is required of the compatibility of the displacements of slabs and all other members; slabs must be capable of resisting the induced stresses.

(f) In design of frames containing wall panels it may be assumed that shears in the panels are in equilibrium with axial forces in the frame members, neglecting tensions in the wall.

Points specified herein refer principally to items that are often forgotten in standard design practice. Paragraph (b) is vague; however, too many variables are involved, so no rule of thumb could be set up to decide when flexural deformations in shear walls and the like are important.

Paragraph (c) contradicts the practice and observations of many engineers, who are willing to consider combined action of steel and concrete even when connectors are not provided. Absence of quantitative information on the subject led to this conservative requirement.

One possible objection to (d) concerns the apparently low factor of safety that it provides against overturning. However, the proposed code allows little reduction in overturning moments. Moreover, actual overturning of structures is not anticipated.

Paragraph (f) is admittedly inexact. Yet no better assumption could be found in the literature. The problem of interaction between frames and the wall panels they encase is complicated by the presence of high localized stresses in the frames, in zones partially confined by the walls.

20. Instruments. Permit for occupancy of structures covering an area in excess of 10,00 m^2 (one million sq ft) or whose height exceeds 45 m (148 ft) necessitates proof before the Building Department to the

effect that deformeters and oscillographs are installed and that these instruments are capable of accurately recording strong motions.

21. Changes to the Present Chapter. Any requirement in this chapter may modified when analytical or experimental information becomes available which makes such changes possible. (The rest of this article concerns administrative requirements for said modifications.)

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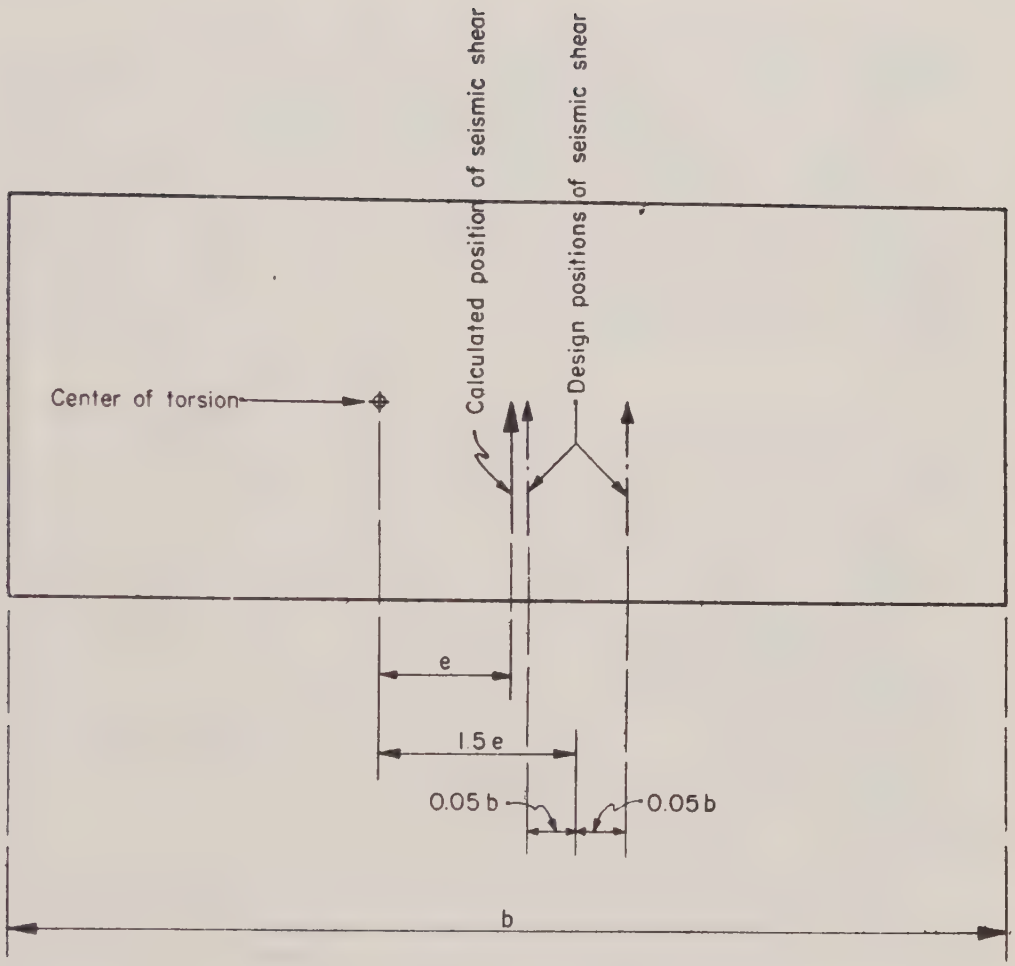

FIG. 1. TREATMENT OF TORSION IN STATIC METHOD

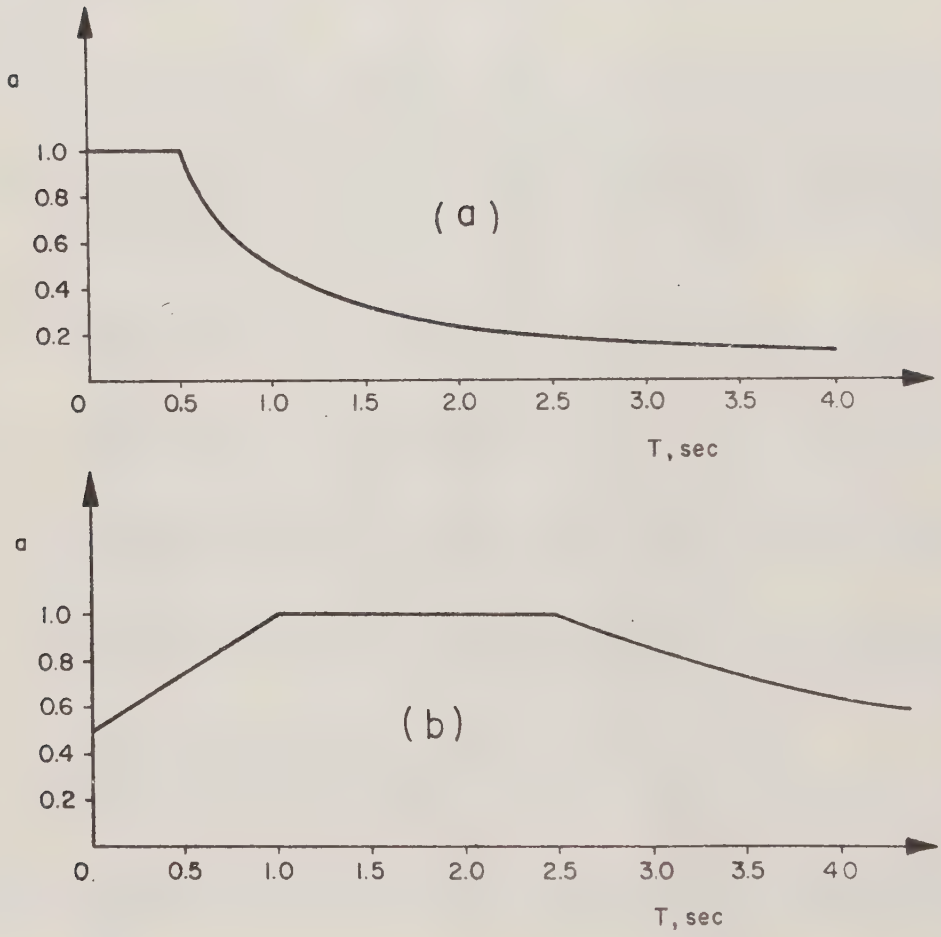

FIG. 2. DESIGN ACCELERATION SPECTRA

THE SEISMIC DESIGN OF A NUCLEAR POWER STATION FOR JAPAN

by

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INTRODUCTION

Japan's first natural uranium nuclear power station is to be built at Tokai - Mura as part of the network of the Tokyo - Yokohama area of Honshu, operated by members of the Japan Atomic Power Company. The J.A.P.C. was formed in 1957 by the collaboration of nine power companies, the governmental Electric Power Development Company, and leading industrial concerns. An Enquiry for a single - reactor station based on the Calder Hall type was issued in February 1958 and tenders were submitted in the following July. The letter of Intent on G.E.C. was issued in April 1959 and the Contract signed in Tokyo on the 22nd December, 1959. Preliminary work by the J.A.P.C. on site and excavation for the reactor has already begun. Concreting of the reactor structure will begin at the end of the year and the station is due for completion in 1964.

Possibly the most interesting feature of the station is that it is designed to resist earthquake, it is not surprising, therefore that this station should differ appreciably in many ways from the one being built at Hunterston in Scotland, since from its initial conception all design decisions were made only after giving due consideration to the effect of an earthquake on the particular part of the station being considered. For example, the choice of bottom charge for the reactor in the case of Hunterston was made after a careful study of all the mechanical advantages and disadvantages associated with this system; however, for the Japanese Station it was clear from the beginning that top charge was necessary because of the importance of keeping the centre of gravity of the reactor building and its contents as low as possible to minimise the effects of horizontal accelerations during an earthquake. Again, a noticeable difference between the Japanese reactor and all the British commercial reactors is that the former has a core with a triangular lattice, while the latter have cores with the familiar square lattice. Although perhaps not apparent at first sight, the reason for this change is due entirely to a consideration of the structural stability of the core during an earthquake. Tests have shown that when their nuclear properties are essentially the same, a core with bricks on a triangular lattice has individual bricks which have a higher ultimate strength than the corresponding bricks used in a core with a square lattice.

Basically the station has been designed as a number of self contained units, each of which is capable of withstanding earthquakes far more severe than those likely to occur in Tokai-Mura. Thus the reactor, heat exchangers and the duct system form one unit integral with the biological shield and the superstructure of the reactor

* G.E.C./Simon-Carves Atomic Energy Group.
+ The British General Electric Company of Japan Ltd.

building, while the turbine hall forms another unit. During normal operation of the station, there is a central pipe and cable bridge giving a relatively flexible connection between these units, however, the reactor building has been provided with all the plant necessary for the complete removal of the heat generated in the reactor core even if all connections with the turbine hall and other buildings and supplies were broken. This provision assumes that the reactor has been automatically shut down and that fission reactions have been stopped so that heat comes only from the radioactive decay of the fission products in the fuel.

In general, all parts of the plant are designed so that their relative displacements during an earthquake do not interfere with the operation of normal controlling devices, such as the insertion of control rods into the core. Nevertheless, to ensure maximum safety, provision is made to meet conditions associated with a catastrophic earthquake having an intensity far in excess of that for which the station is designed, and therefore, far greater than any earthquake ever experienced or ever likely to be experienced in Tokai-Mura. The main essential in any emergency is to stop the fission reactions in the uranium fuel. In case the normal control rods should not be able to do this, an alternative shut down system has been devised for the G.E.C. reactor. This emergency system can be triggered manually, by seismographs, or by special sensing devices that will detect any unsafe misalignment of the core support grid, the core, or the reactor pressure vessel. Shut down is caused by allowing many thousands of small boron-steel balls to fall into a number of special channels in the core. The size of the balls is such that even if the whole reactor were grossly tilted from its vertical position they would still roll in, and even severe disruption of the reactor core would not prevent their entry. Moreover, it is possible to remove the balls from the core after use by means of a magnet grab, or by using the pressure difference of the carbon dioxide gas inside the system. To prevent spurious operation of the system due to circuit faults, the control circuits are triplicated and the simulataneous operation of two is necessary to initiate an emergency shut down.

EARTHQUAKE DESIGN LOADS

As will be appreciated, the additional loads imposed on a structure during an earthquake cannot be predicted accurately; furthermore, the dynamic behaviour of any particular structure depends on such intractable factors as the inherent degree of damping, foundation conditions, and so on. Thus the formulation of design criteria for an earthquake resistant structure is particularly difficult and must be determined from experience and a consideration of structural designs that have been proved safe during past earthquakes.

For some structures, including most of those for the Tokai-Mura nuclear power station, the occurrence of an earthquake can be considered from a design point of view as resulting in additional forces acting vertically and horizontally through the centre of gravity of each component, and proportional to their mass. Thus if W is the weight of a structural component, the temporary horizontal

and vertical earthquake loading would be $C_H W$ and $C_V W$, where C_H and C_V are seismic coefficients given by the building codes of the country concerned.

With regard to the seismic coefficients for the present design, in accordance with a policy of extreme safety, it was specified that.

(1) The reactor building and its foundation mat, except those parts specified under items 2 and 3 below, shall be designed to resist safely a seismic force one and a half times the value specified in the Japanese Building Code.

(2) The biological shielding and principal sub-structures surrounding or supporting the primary circuits shall be designed to resist safely seismic forces three times those specified in the code.

(3) All structures and structural parts inside the biological shielding structure and steam-raising units shall be designed to resist safely horizontal seismic forces as specified in item 2 above, together with vertical seismic forces whose intensity is equal to 50 per cent of the horizontal forces. In this case, the vertical and horizontal forces are considered to act simultaneously.

(4) Seismic coefficients for the gas ducts, joints and supports shall be 2.0 in all horizontal directions.

PARTICULAR EARTHQUAKE DESIGN PROBLEMS.

It is outside the scope of this paper to describe in detail all the problems associated with the design of an earthquake-resistant nuclear power station, for this reason attention is confined mainly to the more interesting structural problems of the reactor core, pressure vessel and gas ducts. However, before considering these particular problems, a brief survey of the general arrangement of reactor building may be of interest.

Referring to figure 1, it will be seen that the whole reactor structure is supported on a cellular reinforced concrete raft, the top face of which is located at ground level, while the bottom face rests on a system of caissons 22 feet below. The caissons have been designed to withstand the horizontal forces due to an earthquake, as well as the normal dead load of the structure. The cellular raft consists of a number of longitudinal and transverse members which mainly carry shear loads, and top and bottom slab members which mainly resist bending. It will be noticed that the cold ducts are partially located inside the cellular raft, an arrangement which leads to a reduction in the overall height of the structure. Furthermore, each cold gas duct is taken from the side of the steam-raising unit remote from the reactor, thus enabling the steam-raising unit to be located much nearer the biological shield, to which it is attached near the top conical head of the vessel by means of an earthquake restraint.

The cellular raft, the biological shield, and the reactor building superstructure up to the roof of the biological shield, form a monolithic unit, while the two box-like structures surrounding the hot gas ducts and supporting the rails for the charge machine have been considered as independent units cantilevered from the top of the biological shield. This is justified on account of their relative flexibility. In accordance with standard Japanese practice for earthquake resistant structures, the service building and other more conventional buildings have a mild steel skeleton embedded in the concrete with an approximate amount of reinforcement. The stack at the top of the reactor building has been designed for normal seismic forces acting in a horizontal direction.

With regard to the structural analysis for the complete reactor building, specified seismic coefficients were used to determine the magnitude of the horizontal shear forces during an earthquake. This shear was then distributed between the various components of the structure in proportion to their stiffnesses, thus ensuring compatibility of displacement in a horizontal direction.

(a) REACTOR CORE

The reactor core consists of a cylindrical stack of graphite bricks supported on a plate girder grillage and contained within a cylindrical steel shell designed to give additional stability to the graphite structure during an earthquake. The core is approximately 12.8 m (44 feet) diameter, 8.15 m (26 feet) high, and, excluding the grillage and stabilising cylinder, weighs over 2,000 tons when fully charged. Except at the boundary of the core, each brick in the moderator and reflector is a hexagonal prism approximately 230 mm (9 inches) across the flats and 940 mm (37 inches) long. There are no tiles ie. thinner layers separating the layers of bricks. As shown in figure 2, keys and keyways are machined on alternate sides of the bricks. Thus, when the core is constructed, there is a complete interlocking of the graphite bricks since the key of one brick is located in the keyway of an adjacent brick (see figure 3).

The central region of the core forming the moderator is approximately 11.2 m (37 feet) in diameter. The bricks in this region are centrally bored, forming a core with a triangular lattice. To maintain complete interlocking of the bricks without reducing locally the structural strength of the core, control rods are located in empty charge channels rather than in specially prepared channels at the intersection point of three bricks. The outermost bricks of the reflector are spigotted to narrow stiffening rings which project from the inside face of the steel earthquake stabilising cylinder which surrounds the core, bricks at the bottom of the core are supported on the steel support plates resting on top of the grillage.

As a result of constructing the core from interlocking bricks, body forces due to an earthquake can be transmitted readily from brick to brick via the keys and hence into the stabilising cylinder and down to the grillage and bottom skirt of the pressure vessel. Furthermore, the geometry of the bricks enables the core to expand symmetrically radially without imposing any type of restraint on the keys. For example,

during start up of the reactor, the thermal expansion of the core stabilizing cylinder and grillage is about three times as large as for the core, to which they are attached by means of the spigots described above. The radial keys ensure that every brick is dragged out a small distance radially by the stabilizing cylinder thus forming a gap of approximately 0.58 mm (0.023 inches) between faces of adjacent bricks and resulting in an overall expansion equivalent to that of steel. The core will be constructed with small initial gaps between adjacent brick faces to accommodate any initial Wigner expansion of the bricks.

A full size model of part of the core which demonstrates this effect is shown in figure 4. In the model the radial spokes are attached to a hub which is loosely located over the central brick, while the ends of the spokes are spigotted into the six corner bricks of the hexagonal portion of core. Six steel sliding bars on the boundary of the model ensure that there is no relative rotation of bricks when the gaps in the model are closed, the hub is located so that the radial spokes are eccentric. Thus, a rotation of the hub into a position which reduces the eccentricity of the spokes produces an equivalent radial expansion of the core accompanied by gaps between individual bricks. For demonstration purposes the hub of the model is rotated by means of the wooden handle shown in figure 4.

Another feature of the interlocked core is that individual bricks can expand or contract relative to their own centre of gravity without experiencing restraint from adjacent parts of the core. For example, during the life of the reactor the graphite is subjected to irradiation damage which is approximately proportional to the fast flux distribution. As a result, after prolonged irradiation every brick tends to shrink, but since the irradiation damage across a key and keyway is the same, there is no change in the initial clearance in the keyway. At the same time the radial keys ensure that the bricks shrink towards their own centre of gravity, thus opening gaps between adjacent brick faces. In practice, even in the most highly irradiated part of the core, this gap will be only about 0.254 mm (0.1 inches) after twenty years.

When considering the stability of the core against a horizontal earthquake force, it is useful to consider initially the outermost ring of bricks fixed to the cylinder. The keys between this ring of bricks and the adjacent ring are radial towards the centre of each brick. Hence, each brick in this ring is fixed relative to the outer ring by virtue of the inclination of the keys. Similarly, the third ring of bricks is fixed to the second, and so on to the centre of the core. Thus the degree of fixity between any two bricks depends on the slackness in the keys and keyways, and since this can be controlled by small machining tolerances, it is found that the total slackness in the core is quite small.

In general, it may be observed that whilst the radial keys permit temperature and shrinkage movements of bricks without restraint, they resist non-symmetrical distortion of the core as a whole, except in so far as there is a very limited amount of distortion due to clearances between keys and keyways. At the same time, and this has been demonstrated by a test, it is impossible to jam a key in a

keyway either by displacement or rotation of a single brick.

As previously explained, the earthquake loads are transmitted as shear across the keys of individual bricks to the steel stabilising cylinder. Calculations and strength tests on individual bricks show that provided the load is transmitted smoothly through the core there is a large factor of safety on the structural strength of the core even when the horizontal earthquake load is taken as high as 0.8g. Two factors ensure a smooth distribution of boundary reactions of the core. Firstly, the steel stabilising cylinder and core are connected by relatively flexible spigots which prevent load concentrations. Secondly the core consists of thousands of unit bricks which during an earthquake tend to distribute the load more smoothly than the case of a core made up of a small number of large monolithic units, each with a corresponding large number of keys and keyways.

Experimental work on the earthquake stability of the core is being carried out at the Japanese Building Research Station in Tokyo. For this purpose a large vibration table has been installed so that horizontal slices of the core as much as 3.55 m. (14 feet) diameter can be given accelerations greater than those expected to be experienced during an earthquake. For example, at the time of the Kanto District Great Earthquake in 1923, the horizontal acceleration caused in the ground on the hill-side of the City of Tokyo was in the order of 0.1g. Accelerations far in excess of this value can be given to models of the core weighing up to 28 tons by using the above vibration table. Furthermore, the table can be tilted mechanically into a near vertical plane so that static loads equivalent to 0.7g. can be imposed on the graphite bricks and so their behaviour can be studied when under the action of the specified earthquake load.

In general the work being done in Japan is confined mainly to large diameter slices of the core using bricks having an almost full size cross-section and an axial length of 3 inches. Such tests have been designed to study the distribution of the earthquake load in the core, the interaction of loads between the core and the steel stabilising cylinder, as well as the radial expansion of the core during temperature changes.

Additional experimental work on the earthquake behaviour of the reactor core is being done at The General Electric Company's works in England, and for this purpose a reasonably large vibration table has been constructed. This table has been designed to carry three dimensional models of the core about 1.84 m. (6 feet) diameter and 1.92 m. (3 feet) high, and by suitably choosing the springs in the machine it is possible to develop accelerations of 2g with amplitudes of 50.8 mm (2 inches).

For the particular experiments being done at Erith, over 2,500 graphite bricks 76. mm (3 inches) wide and 152 mm (6 inches) long are being used to construct a representative portion of the core. With this model it is proposed to study the distribution of the earthquake forces from one horizontal slice of the core to another, and hence determine the distribution of the load transmitted to the grillage and to the core stabilising cylinder. The vibration table will also be used to consider the degree of damping in the core due

to this non-monolithic unit brick construction.

(b) REACTOR PRESSURE VESSEL AND SUPPORTS.

The reactor pressure vessel (see figure 1) is a sphere of fine-grain aluminium-killed steel having a diameter of 18.9 m (62 feet) and a thickness of 83 mm (3¼ inches); its gross weight is about 1,500 tonnes, and it has been designed to withstand an internal pressure of 16.2 kg/cm^2 (230 lb/in^2). The vessel is supported by a cylindrical skirt having a diameter of 12.4 m (40 feet 9 inches) and a thickness of 63.5 mm (2½ inches). During normal operating conditions this skirt supports the combined weight of the pressure vessel, reactor core, grillage, etc. (approximately 4,000 tonnes), while in the event of an earthquake it resists vertical oscillations corresponding to an additional dead load of approximately 1,300 tonnes.

The skirt is extended several feet into the vessel in order to form a support for the plate-girder grillage which carries the reactor core. The actual length of the internal skirt was chosen to be as short as possible, consistent with the condition that its length was sufficient to permit substantial attenuation of applied edge bending and shear stresses.

In addition, there is a further skirt located on the outside of the vessel above the equator, as shown in figure 1. This skirt has a thickness of 25.4 mm (1 inch), and has been designed to transmit a portion of the horizontal earthquake load from the reactor pressure vessel to the biological shield. This is effected by means of two thin steel diaphragms located in horizontal planes at the upper end of the skirt and anchored to the inner face of the biological shield. The diaphragms have sufficient flexibility to permit vertical temperature movements of the top skirt without imposing serious restraining forces on the vessel.

The effect of supporting a spherical pressure vessel and its core on continuous skirt supports, one of which is located inside the vessel and the other directly underneath on the outside, is to induce in the skirt and loaded vessel a system of bending and shear stresses due to the constraining effect of the skirts. The magnitude of these stresses depends on the relative thicknesses of the components joined at the discontinuity as well as the pressure in the vessel, the weight of the core, the temperature variations in the structure in the region of the discontinuity, and the intensity of any particular earthquake.

For the design of the J.A.P.C. reactor pressure vessel, a detailed stress analysis was made for the supporting means, and it was found that by suitably choosing the thickness of the various skirts, the bending stresses induced in the structure could be kept relatively low so that the sum of the membrane and bending stresses in the vessel and skirt at the point of discontinuity were everywhere reasonably less than the yield stress of the material even during a severe earthquake.

Basically, the analysis of the skirt problem reduces to the finding of the unknown bending moments and shear forces induced in the four components of the structure meeting at the

discontinuity. This involves the solution of a large number of simultaneous equations which, for the J.A.P.C. pressure vessel, was done to give expressions for the induced stresses in terms of the following dimensional variables.

(1) thickness of vessel

(2) thickness of skirts

(3) diameter of skirts

(4) diameter of sphere

For design purposes this solution was used to find expressions for the stresses induced in the vessel and skirts due to the following systems:

(1) internal pressure in vessel

(2) weight of core

(3) weight of vessel

(4) weight of standpipes and other vessel attachments.

(5) an axial temperature gradient in the bottom skirt.

In addition, the restraining effect of the top skirt was considered when determining the induced stresses due to horizontal earthquake forces.

With regard to the stress analysis for the skirt to shell junction, a method for analysing the induced bending and shear stresses in a symmetrically loaded shell of revolution was developed in 1912 by H. Reissner[1]. This method was generalised and applied to particular cases by E. Meissner[2] who obtained a solution to the basic differential equation in the form of hypergeometrical series. Later an approximate solution to the problem was obtained by J.W. Geckeler[3], who made the simplifying assumption that in the region of the induced moments and shear forces, the shell of revolution can be replaced by an equivalent cylinder. A better approximation, and the one often used in practice, was obtained by Hetenyi[4] in 1938. This approximation is equivalent to assuming that in the region of the vessel where the induced stresses are relatively large, the vessel can be replaced by an equivalent conical section, because the meridional length of such a region is small compared with the radius of curvature of the vessel at the point considered.

Based on the approximate theories of Geckeler and Hetényi, a general analysis for the normal operating stresses at a skirt to shell junction has been done by Hicks[5], and this was used to evaluate the stresses for non-earthquake conditions in the J.A.P.C. reactor vessel.

For a non-symmetrically loaded shell of revolution, various methods of analysis have been proposed by different authors. In all

cases a number of assumptions were made in order to reduce the basic differential equations for the shell to forms which considerably simplified the mathematics. Thus, for a cylinder, the basic equations of Flügge[6] are the most accurate, while the equations of Donnell[7] although less accurate are easier to apply to particular problems and can be used to give a closed solution which has been obtained by Hoff[8]. Similarly, for a sphere, basic equations have been obtained by Havers[9], Tsuboi and Akino[10], and others.

For the non-symmetrical earthquake effects on the reactor pressure vessel and skirts, a complete analysis was done by Leckie and Levesley (unpublished) of Cambridge University. Using the equations of Havers for the sphere and Hoff for the skirts, they obtained values for the earthquake stresses induced into the structure at the point of the bottom skirt support. They found that during an earthquake the horizontal shear on the reactor core is carried mainly by the bottom skirt, while the horizontal shear on the pressure vessel due to the top dome standpipes is transmitted almost completely to the biological shield via the top skirt and diaphragms. On the other hand, the horizontal earthquake body forces on the sphere are shared more or less equally between the top and bottom skirts.

It is interesting to note that in the event of no top skirt, the induced earthquake stresses at the bottom skirt to shell junction are increased considerably. This increase can be attributed mainly to the additional overturning moment in the bottom skirt by the standpipes located at the top of the reactor pressure vessel.

It will be appreciated that for a reactor pressure vessel the analysis for the stresses at the junction of the vessel and support skirt involves a great deal of numerical computation, for example; sets of more than thirty simultaneous equations have to be solved to determine the magnitude of the earthquake stresses. For this reason, where possible, experimental work has been done to verify theoretical calculations and to justify the use of particular shell theories. At the works of the General Electric Company, a one sixth scale model of the sphere and support is being used to check earthquake stresses, while a full size mock-up of part of the support skirt is used to determine temperature gradients for the evaluation of thermal stresses. Furthermore, additional experimental work is being done independently in Japan under the direction of Professor Muto for the Japan Atomic Power Company.

(c) MAIN GAS DUCTS.

Consideration of such features as mass flow, frictional losses and the number of steam raising units, led to a final duct diameter of approximately 6 feet. To ensure flexibility for temperature expansions, three hinged bellows are incorporated in each duct system, the bellows being designed to take an axial load without transmitting any appreciable bending moment.

As previously stated, during an earthquake the ducts are considered as under the action of additional loads equivalent to 2g acting horizontally and 1g acting vertically, at the same time, the vibration behaviour of the system must be such as to ensure that

there will be no danger of excessive resonance displacements due to forced vibration caused by an earthquake. Since each duct system is co-planar, deflections in directions normal to the plane of ducts have been restricted by introducing a series of sliding supports which merely permit axial and diametral thermal expansions, while vertical and horizontal earthquake deflections in the plane of the system are restricted by a number of viscous dampers each of which is connected to a duct by means of a single lever. The function of the lever is to improve the effectiveness of the damper by increasing its throw for a given deflection of the duct at the point where it is attached to the lever. In general it has been found that the deflections of the duct system are small and correspond to the lower limit of movement normally associated with effective viscous damping. In this respect it is advantageous to have lobster-back bends in the ductwork to increase its overall flexibility.

To illustrate the method of design, reference may be made to the particular case of the hot duct system shown diagramatically in figure 5. As will be seen, the duct is attached to the reactor pressure vessel and the heat exchanger at the points A and C respectively. The hinged bellows are located at B_1, B_2 and B_3, while a valve which isolates the pressure vessel from the heat exchanger has been placed at the point V. The total mass of the bellows and valve is in the same order as the mass of the complete duct system from A to B.

An initial calculation showed that in the event of an earthquake the duct system became seriously over stressed unless supported at a number of intermediate points along its length; furthermore, mainly because of the presence of the bellows, gross displacements where likely to occur due to forced vibrations. For this reason additional horizontal supports were placed at the points H_1 and H_2, and additional vertical supports were placed at Z_1 and Z_2, these latter supports being designed so as not to support any of the dead load of the ducts during normal operating conditions. Since considerable movements occur in the ductwork due to temperature changes in the reactor during normal operating conditions, and because these movements are relatively slow compared with the frequency of vibration of an earthquake, the additional support for the ductwork was supplied by using viscous dampers at the intermediate points mentioned above. As will be appreciated, in the event of an earthquake, the deflections of these supports will depend on the periodicity of the applied loading, the natural frequency of the ductwork, and the viscosity of the dampers themselves. It follows that within practical limits this last factor may be varied to give a high or low resistance to the deflections of the ductwork as required.

With regard to the design of the dampers, particularly with reference to their viscosity, this was done by considering the allowable stress and corresponding deflections in the ductwork during an earthquake. For example, if there is full restraint at H_2, a horizontal earthquake loading will result in certain stresses in the duct system but there will be no deflections of the point H_2. However, if the restraint H_2 is reduced, there will be an accompanying deflection of the ductwork at this point, with the result that additional stresses will be induced in the duct system. This implies

that if the maximum stress in the system is initially fixed, then this determines the maximum deflection at H_2 and the viscosity of the damper.

The above example has been simplified considerably to illustrate the essential features of the design calculations. In practice it was necessary to consider the simultaneous deflections of all the damper supports during an earthquake, and to adjust the viscosity of the various dampers to prevent excessive stresses in any part of the system. Furthermore, since the deflections of the restrained duct system are inherently small, it was necessary to consider the additional flexibility due to the lobster back bends and the localised flexibility of the reactor pressure vessel at the point S, the latter being estimated by using reference 11.

Since the effectiveness of the dampers depends on their actual location on the ductwork it was essential to study the vibrational behaviour of the system and see where the deflections became excessive, particularly near the point of resonance. For this purpose models of the ductwork were tested on a small vibration table having a frequency range of two to twenty two cycles per second. The models were made geometrically similar to the prototype in the direction of the axis of each straight component of the duct systems, and, although the models were made from flat brass strips rather than tubes, the correct bending stiffnesses were maintained. Masses were added at the valve and bellows positions to obtain the correct ratio of concentrated to distributed load, while very thin short lengths of spring were used to represent the bellows.

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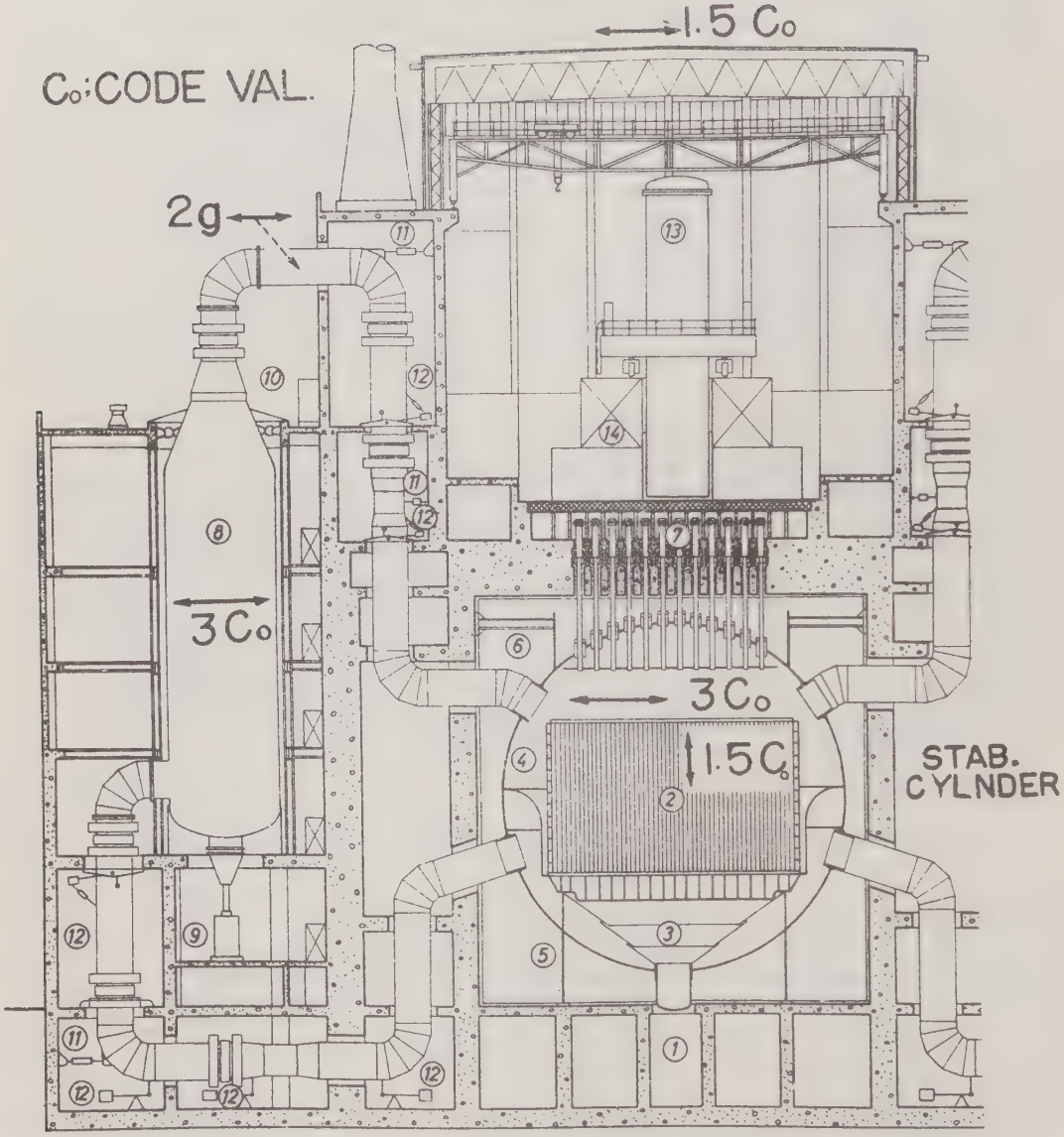

1. Reactor Raft
2. Reactor Core
3. Debris Collector Cones
4. Pressure Vessel
5. Reactor Support Skirt.
6. Pressure Vessel Earthquake Restraint.
7. Charge/Discharge Standpipes.
8. Steam Raising Unit
9. Gas Circulator Turbine Drive
10. S.R.U. Earthquake Restraint
11. Duct Earthquake Restraint
12. Duct Counterweight Supports.
13. Charge Machine
14. Charge Machine Bridge.

FIGURE 1.

CROSS SECTION THROUGH REACTOR BUILDING

FIGURE 3

SLICE OF A MODEL GRAPHITE CORE SHOWING INTERLOCKING OF BRICKS.

FIGURE 2

MODEL OF HEXAGONAL GRAPHITE BRICK SHOWING KEYS AND KEYWAYS.

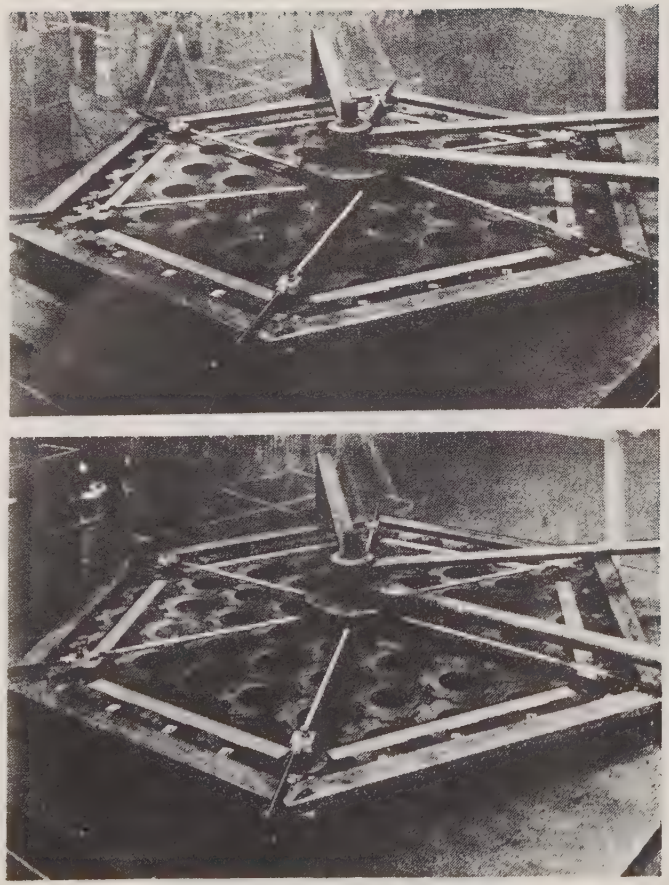

FIGURE 4

MODEL FOR DEMONSTRATING UNRESTRAINED RADIAL EXPANSIONS OF THE CORE.

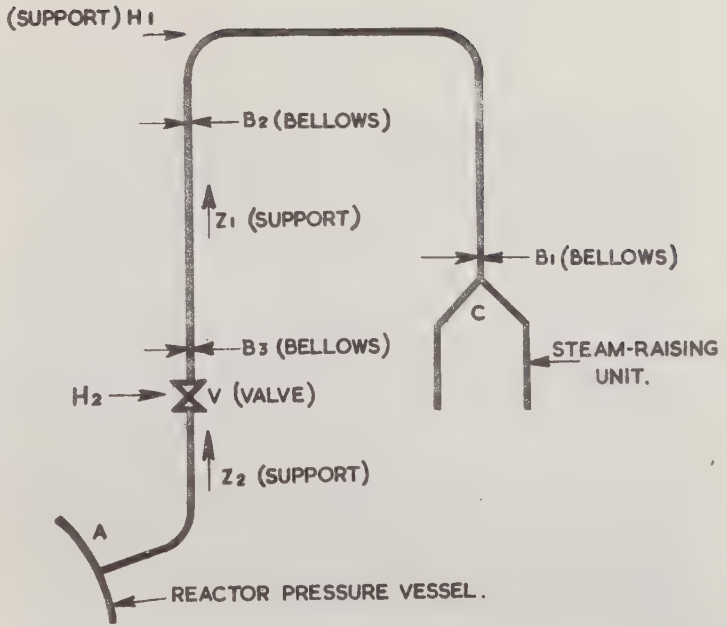

FIGURE 5

DIAGRAM OF HOT DUCT SHOWING POSITIONS OF EARTHQUAKE SUPPORTS.

STUDY ON THE EARTHQUAKE RESISTANT DESIGN OF GRAVITY TYPE DAMS

by

MOTOHIRO HATANAKA*

INTRODUCTION

For the earthquake resistant design of gravity dam, the method now adopted is based on the principle that no hair cracks should be allowed to occur. Accordingly, it may be approved that the behavior of dam subjected to the seismic motion is to be dealt with as an elastic vibration. This paper describes firstly that the fundamental natural period and mode of dam vibration, which are the basis of determining the seismic coefficient for design, are approximately expressible as those of bending-shear vibration by the beam theory, and also clarifies that in some high dams in Japan the values of predominant period of earthquake motion, the fundamental natural period of dam and resonant period of hydrodynamic pressure are very close to each other.

The vibration of dam in the case where the small number seismic waves having the period described above act suddenly, is also solved approximately, and the author's principle of earthquake resistant design is clarified in this report.

As the conclusion so far obtained, though it is not sufficiently perfect and further study is expected, a method of earthquake resistant design is proposed by the author.

SEISMIC COEFFICIENT OF DESIGN

In the present earthquake resistant design of gravity dam, the seismic coefficient of constant magnitude and of uniform distribution, which is based on the same static consideration as what has been taken in the designing of other common structures, is used. The rational seismic coefficient based on the so-called "method of seismic coefficient" (the method of lateral force coefficient) is obtained by dividing the lateral force of such magnitude by the weight of structure as to give the same amount of deflection as that of elastic structure caused by the seismic motion. (1),(2). And the seismic coefficient can be expressed by the sum of those of normal modes of vibration of each order.

In general, however, the normal mode having the nearly equal period to that of seismic motion is most influencial.

In a high dam, the predominant period of seismic motion of the dam foundation is nearly equal to the fundamental natural period of dam, and therefore, it seems quite sufficient that only the seismic coefficient for the fundamental mode is taken into account in an approximate solution. Accordingly, the seismic coefficient K is expressed by,

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as one-dimensional problem

$$K=\frac{1}{g}CYA$$

$$A=\frac{n^2}{\omega}\int_0^t e^{-\varepsilon(t-\tau)}\alpha(\tau)\sin\omega(t-\tau)\,d\tau, \qquad \text{.....(1)}$$

and as two-dimensional problem

$$K'=\frac{1}{g}C'XYA'$$

$$A'=\frac{n'^2}{\omega'}\int_0^t e^{-\varepsilon'(t-\tau)}\alpha(\tau)\sin\omega'(t-\tau)\,d\tau. \qquad \text{.....(2)}$$

The distribution of seismic coefficients are in direct proporation to Y and X, Y, and hence the absolute value of them are determined by A and A'. To obtain the absolute values of K and K' it is necessary to know about $\alpha(t)$. As to $\alpha(t)$, many cases such as obtained by investigating the seismograms of strong earthquakes in the past, (3); by assuming the mean velocity of ground motion as a constant and adopting one of the resonant waves having the largest velocity (4); and by considering one sine (5) or cosine (6) wave, etc. have been discussed. At any rate, the most important data for determining the absolute value of seismic coefficient are the actual behaviors of dam and the ground motion which were caused by strong earthquakes in the past. At present it seems that there is no alternative but to determine, the above-mentioned coefficient by our technological judgement based on past experiences. Hence, in the following chapter, the fundamental natural periods and the modes of dam are discussed.

THE FUNDAMENTAL NATURAL PERIODS AND THE MODES

As to the vibration of gravity dam, Dr. Hatano (7) discussed each of the bending and shear vibrations as seperate stational vibration, for the fundamental triangular section of asymmetrical form. However, as already pointed out by the author (2), the influence of shearing force is considerably large in the dam having the ratio of base width b to height h 0.8~0.9, such as in the gravity dam, and therefore the bending-shear vibration considering not only the bending moment but also shear, must be analysed. The fundamental natural period T_{BS} of bending-shear vibration is, in geheral, given by

$$T_{BS} = C_{BS}h\sqrt{\rho/E}\,. \qquad \text{.....(3)}$$

Also as well known, the fundamental natural periods of bending vibration T_B and that of shear vibration T_S are expressed by the following equations respectively:

$$T_B = 4.098(h/\alpha)\sqrt{\rho/E} = C_B h\sqrt{\rho/E}\,, \qquad \text{.....(4)}$$

$$T_S = (2\pi h/2.4048)\sqrt{\rho/E} = C_S h\sqrt{\rho/E}\,. \qquad \text{.....(5)}$$

The variations of C_{BS} depending upon the slope of dam section α and Poisson's ratio σ are shown in Fig. 1. a). In the figure, values of coefficients C_B , and C_S in eqs. (4)(5) are also shown.

T_{BS} tends to T_B when α becomes smaller, but on the contrary, to T_S when α becomes large. As clearly seen in the figure, for such a slope of $\alpha = 0.9 \sim 1.3$ as found in the gravity or hollow gravity dam, the value of T_{BS}/T_B is $1.3 \sim 1.6$ for $\sigma = 0.15$, it may be easily understood that the effect of shearing force is fairly large.

Therefore, the vibration of dams must be considered not as the bending vibration but as the bending-shear one.

Then, Fig. 1. b) illustrates an example of the fundamental mode Y_{BS} of bending-shear vibration. For $\alpha = 0.9$ or so the normal mode is of bending-shear vibration, and is far apart from Y_B .

And, for $\alpha = 1.5 \sim 2.0$ the mode of bending-shear vibration Y_{BS} is of nearly straight line. The value of Y_{BS}/Y_B at the middle of dam height is $1.7 \sim 2.1$ ($\sigma = 0.15$) for $\alpha = 0.9 \sim 1.3$, and is considerably larger than that of T_{BS}/T_B. The variation of modes Y_{BS} with that of the Poisson's ratio σ is very small and for $\sigma = 0.15 \sim 0.45$ at most 7 percent or so. Fig. 1 is shown for the dam with symmetrical triangular section. However, for such values of α as seen in the gravity and hollow gravity dams, the error that might be introduced into the above analysis due to the asymmetry of dam section is probably less than several percent.

Accordingly, the above figures may be used for the practical purposes with technologically sufficient accuracy.

According to some of the papers reported on the measurements of vibration of actual dams, it may be pointed out that measured periods* are remarkably larger than those calculated by assuming as the bending or bending-shear vibration. (9). One cause of this fact is said to be the effect of the deformation of dam foundation. (9)(10). For simplicity, the vibration of dam will be discussed in the case where the dam and the foundation can be expressed by the one-mass system as shown in Fig. 2.

In this case, the natural period of dam with rocking motion T_r becomes as follows (8):

$$T_r = T\sqrt{1 + \eta/\delta} \quad . \qquad \cdots\cdots\cdots\cdots(6)$$

As clearly shown by eq. (6), in the structure which is attended by not only the elastic deformation but also the displacement due to rocking motion, the natural period becomes as long as the value $\sqrt{1+\eta/\delta}$ multiplied by the natural period T obtained in the case of considering the elastic deformation only, neglecting the deviation by rocking motion. Then, the seismic coefficient for design K_r in the above case is expressed by

$$K_r = \frac{\eta_r}{g}\int_0^t \alpha(\tau)\sin \eta_r(t-\tau)d\tau. \qquad \cdots\cdots\cdots\cdots(7)$$

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* Measured period of Miura and Taifu Dam in Table 2.

On the other hand, the seismic coefficient K of the structure of one-mass system, when only the elastic deformation is taken into account, is obtained by putting $C=\gamma=1$ and $\varepsilon=0$ in eq. (1),

$$K=\frac{n}{g}\int_0^t \alpha(\tau)\sin n(t-\tau)d\tau . \qquad \text{.....(8)}$$

Comparison of eq. (7) with eq. (8) shows that eq. (7) is equal to eq. (8), when putting $n=n_r$. Therefore, as will be understood by the concept of acceleration spectrum (or seismic coefficient spectrum), it may be allowed to use n_r (or T_r) instead of n (or T). That is to say, even the case where the displacement due to rocking motion is accompanied, the acceleration spectrum (or seismic coefficient spectrum), is of similar form to that obtained in the case of the elastic deformation only, and does not show any particular change in the form due to rocking motion.

EFFECT OF THE RESERVOIR

(1) Hydrodynamic pressure working on vertical wall plane

The fundamental theory of hydrodynamic pressure was established by Dr. H.M.Westergaard (11), and was improved by T.Hatano (12) and by S.Sato. (13). Lately S.Kotsubo discussed mathematically the effects of the topography, the dam shape and the shape of contact surface (exposed surface) of dam and water (14) on hydrodynamic pressure, as the problem of unsteady state.

The approximate formulas by Westergaard are very conveniently used technologically, but have some unreasonable aspects.

And furthermore the studies pursued by the latter researchers are reasonable but the calculations based on the studies are some what complicated for the practical use. Therefore, the author attempted to derive approximate formulas which are covenient for practical use, based on the studies made by the above-stated researchers in the following way. As it is well known, when the seismic motion is expressed by $a\sin(2\pi/T)$, and $T \fallingdotseq 1.3$ sec; the hydrodynamic pressure working on vertical wall plane is expressed as follows from Westergaard's formula for steady state:

$$p=\frac{7}{8}kw\sqrt{hy} . \qquad \text{.....(9)}$$

Eq. (9) is applicable when a rigid wall oscillates uniformly along the direction of stream flow. When the rigid wall makes rocking motion at the middle of dam bottom, the hydrodynamic pressure p' is given by

$$p'=\frac{3}{8}k'w\sqrt{hy} . \qquad \text{.....(10)}$$

When the seismic motion occurs suddenly, eq. (9) is rewritten as follows:

$$p=C_0\frac{7}{8}kw\sqrt{hy} . \qquad \text{.....(11)}$$

For example, the coefficient C_o in eq. (11) is about 1.5, when the ground motion of a half-wave of sine form having the same period as the natural period of dam occurs suddenly.

(2) The hydrodynamic pressure on the slope

Eqs. (9)(10) can be used approximately, even safely, for the gravity dam having the steep upstream surface, but they give over-estimated values for the hollow gravity dam having the gentle slope surface unless the effects of slope are considered.

C.N.Zanger published the paper concerning experimental investigations in the steady state on this problem. (15). When the hydrodynamic pressure on the vertical plane obtained by Zanger's equation equals to that calculated by Westergaard's approximate eq. (9), a new multiplier coefficient 0.9 must be used for eq. (9).

Therefore, the hydrodynamic pressure p_s on the slope, when the seismic motion occurs suddenly, can be expressed as follows:

$$p_s = C_o C_m \frac{6.3}{8} k w \sqrt{h y} \qquad \text{.....(12)}$$

(3) The effects of topography

The hydrodynamic pressure obtained above is solved in one-dimensional analysis, but it should be analysed as two-dimensional problem for actual dams. The problem has been discussed by Kotsubo who shows that the resonant period T_w' of hydrodynamic pressure can be approximately expressed by

$$T_w' = T_w \sqrt{h'/h} \qquad \text{.....(13)}$$

Furthermore, according to his investigation, the distribution shape of hydrodynamic pressure is not so distinctly different even when the topography changes greatly, but the absolute value depends upon the form of exposed water area and decreases in the following order: rectangle, semicircle, quadrant and triangle. When the period of ground motion is 1.0 sec, the hydrodynamic pressure for a triangle of exposed water area is about 0.7 times as large as that for a rectangular exposed water area.

COMPARISON OF THE MEASURED VALUES ON THE MODEL AND ACTUAL DAMS WITH THE CALCULATED ONES

In the preceding chapter, the results of calculation are described when the vibration of gravity type dam is assumed as the bending-shear vibration of a symmetrical triangular-section-dam.

In the following, in order to examine the propriety of the author's theory, they are compared with the experimental results obtained by himself, and with the actual measurements performed by Dr. Kishinoue, Dr. Hatano and author, etc., on the actual gravity dams.

(1) Experiment by agar-agar models (16)

The resonant periods and the deflection curves at resonance caused by the shaking table are to be assumed as the fundamental natural periods and the modes of models. In Fig. 3. a), the

relations between the slope of dam model and the fundamental natural periods are represented. The measured value of period shows some deviation from the calculated T_{BS} curve, but the said deviation is to be found rather far smaller as compared with the difference between the above-stated measured value and the calculated T_B curve. Fig. 3. b) shows the fundamental modes.

In the case of $\alpha=2$, the measured values show good agreement with the calculated ones; in the case of $\alpha=1$, the measured values lie between the calculated Y_{BS} and Y_B curves, and in the case of $\alpha=0.75$ the measured values coincide with Y_B curve.

(2) Experiment by hollow gravity rubber models (17)

Table 1 shows a certain hollow gravity rubber model dam the scale of which is 1/200. The model dam is composed of ten elements and of four elements with different heights as shown in Table 1.

The natural period and the mode of the elements are obtained from the resonant period and the deflection by the forced vibration caused by the shaking table. In this case the coefficient of the period of bending-shear vibration (the coefficient C_{BS} in eq. (3)) is 6.296 from Fig. 1. a). Calculations are made by using the above value and they are shown in Table 1 in company with the measured data.

As cleary seen from Table 1, the calculated values show good agreement with the measured ones. The mode of the model is represented in Fig. 4 in which the marks • show the measured ones and the full-lines show the calculated values. The measured values lie between the bending-shear vibration curve Y_{BS} and shear vibration curve Y_S .

(3) Measured periods of actual dams

Table 2 shows the measured periods of a few actual gravity dams. In Taifu and Miura Dams, as it may probably be due to the difference in the vibration methods, the periods are cosiderably longer than those expected by the calculation. The deformation of foundation rock was considered as discribed before, as a cause of this discrepancy, but as seen from eq. (5), $T_r=1.41T$ and $T_r=1.73T$ even for $\eta=\delta$ and $2\eta=\delta$, in other words, the period becomes longer by about 40～70 percent. Therefore, this way of thinking cannot explain the remarkable discrepancy between the calculated results and the measured ones. On the contrary, in Tsukabaru Dam the mesured data agreed well with calculated value T_{BS} . The measured period of Tsukabaru Dam was obtained by the oscillator attached in the middle of the span especially on the top of the dam, and so it may be considered that it expresses accurately the natural period. So, some doubts arise whether the measured values on these dams represent their natural period or not.

(4) Two-dimensional vibration of dam

Although the dam must be substantially treated as a two-dimensional structure, the analogy from the author's studies (2) made on the earth dam suggests that one-dimensional treatment Fig. 1. a),b) may be used correctly enough for the dams having the ratio of the length to the height as given in Table 2.

Fig. 4. a), b) show the two-dimensional modes of Tsukabaru

Dam and of the hollow gravity dam of rubber model, but in the latter model dam, the above-mentioned elements were pasted up to each another so as to form one body. As clearly seen from Fig. 4, the measured values show good coincidence with the calculated ones.

(5) Vibration characteristics of dam in the case of full reservoir

Only a few data are available which provide the results of measurements taken for both full and empty cases of the reservoir formed by one of the actual dams. Therefore, the effects of stored water are considered by using the measurements made on Tsukabaru Dam and the models of hollow gravity dam. The natural periods measured on the rubber model dam for the full reservoir are shown in column 5 of Table 1. Coupled oscillation of dam and hydrodynamic pressure is considerably difficult to be treated with.

Accordingly, by aiming at the fundamental natural period of dam only, and also by considering that the hydrodynamic pressure is a virtual mass added to the dam, the vibration of dam is analysed as that of one-mass system. The vibration amplitude of the top of the model dam in resonance becomes about 10 times as large as that of the ground motion, and the deflection curves in resonance are not so different as those obtained for empty reservoir.

Therefore, as the simplest case by using eq. (10) which gives the hydrodynamic pressure when a rigid wall takes the rocking oscillation centered on the foundation, the virtual mass ΔM which is to be added to the one-mass system having the thickness d can be calculated as follows:

$$\Delta M = 0.218\, C_m w h^2 d/g \,. \qquad \text{.....(14)}$$

Column 6 of Table 1 points out the natural period for full reservoir, calculated by using ΔM in eq. (14), the mass and the natural period of the model dam for empty reservoir. Comparison shows that the simple calculation described above gives fairly good approximation to the measured values. So, by using the hydrodynamic pressure where the deformation of dam is taken into account, as a density function and by computing the natural period by the energy method, better approximation is obtenable.

The natural period and mode of Tsukabaru Dam shown in Table 2 and Fig. 4 are obtained for full reservoir. The natural period for empty reservoir, computed from the period 0.146 sec in Table 2 by using the above method, is 0.134 sec. This shows that stored water elongates the period by 9 percent. These values are obtained when the rigid wall takes rocking oscillation. If the rigid wall takes parallel oscillation, the elongation of the period as discussed above is 22 percent which is obtained from calculation under the same assumption. Calculation performed by Kotsubo by assuming as the coupled oscillation of dam and stored water, gives that stored water elongates the period by 15 percent, and it coincide with the mean value of the above two calculated ones by the author.

For the model test the law of similarity is very important. In this test using a rubber model having a small value of modulus of elasticity, the variation in the mode due to stored water is

not so large. In addition, the deflection curve for Tsukabaru Dam in resonance, as shown in Fig. 4, is in good accordance with the mode γ_{BS} of bending-shear. Hence, it may be considered that the destribution of seismic coefficient for design is expressed by γ_{BS} curve for both full and empty cases of the reservoir.

DEFORMATIONS ON THE TRANSIENT VIBRATION OF DAM

According to the data ever measured in Japan, the predominant period of seismic motion at dam site is 0.1～0.3 sec, and, on the other hand, the fundamental natural period of high dam lies in the range as shown by Fig. 5. Therefore, it can be imagined that the dam resonates to a small number of waves -one to a few waves- of the seismic wave, and that it is under the maximum stress condition.

According to the author's investigation by the mechanical analyser with multiple mass (23), if the seismic motion of sine or cosine type the period of which is the same as the fundamental natural period of dam occurs suddenly, the deformation of dam after the ceasing of seismic motion, may be considered approximately as the fundamental mode γ_{BS} and the deformation caused by that time is close to γ_B rather than to γ_{BS} . The deformation of gravity dam can be approximated by the fundamental mode γ_{BS} for both cases of full and empty reservoir. Therefore, the distribution of seismic coefficient of design is also expressed by γ_{BS} .

However, when taking into account the present method of assuming the uniform distribution of seismic coefficient, the triangular distribution along the height and the sine form distribution along the length can be assumed. All described above is discussed for a case where the acceleration of ground motion which is uniformly distributed along the length of dam, acts along the stream flow direction. But, if the structure is long like the dam, the phase difference between the ground motions at both shores becomes a problem. Therefore, when considering such a seismic motion as an object, it is necessary to take into consideration the seismic coefficient for design the distribution of which is of sine type as shown in Fig. 6. b) curve, (24). To determine the absolute value of seismic coefficient, it is necessary to know the damping characteristics of dam besides the natures of seismic motion itself.

As to the seismic motion, our country has not had such perfect standards of seismogram as those obtained by San Fransisco Committee. As previously described, it may be imagined that the high dams in Japan are expected to be subjected to the seismic motions frequently the periods of which are very close to the fundamental natural periods of dams themselves, and in these cases the duration time of the sesimic motion, that is, the number of resonant waves is a problem. Further investigations have to be made on the seismic motion, but the problem is ascribable to the point how much value obtained by multiplying the seismic coefficient of ground motion is to be used for the seismic coefficient for design. For example, if the seismic motion having a half sine-wave with the same period as that of dam acts suddenly, the multiplier is 1.77 for no damping force $\varepsilon=0$ (natural circular frequency of dam n/ circular frequency of ground p =1.620).

Then, as to the damping coefficient, only a few measured data are available now except the values as given in Table 3.

Furthermore, all described above such as the predominant period of seismic motion, the damping coefficient, etc., were obtained by the experiments of micro-seismic motion or micro-vibration, and hence the values in the case of strong seismic motion should be clarified by further studies. The vibration of dam in the case of full reservoir is very complicated, and the natural period of dam in the case is about 15 percent longer than that in the case of empty reservoir. As illustrated in Fig. 5, the resonant period of hydrodynamic pressure is generally longer than the natural period of dam. In some cases, as pointed out by Kotsubo, the period of hydrodynamic pressure is 1~0.7 times as short as that of one-dimensional hydrodynamic pressure dependent on the topography, and, on the other hand, the natural period of dam becomes 1~1.7 times as long as the said value, due to the deformation of rock foundation. Therefore, the case can be assumed as the critical (most dangerous) condition, where the following three values: the period of seismic motion, the natural period of dam and the resonant period of hydrodynamic pressure coincide with each other.

If the seismic motion of a half sine-wave having the same period as the resonant period of hydrodynamic pressure, acts suddenly, the hydrodynamic pressure working on the vertical rigid wall is about 1.5 times as large as that obtained by Westergaard's approximate eq. (9), and, in an example of natural seismic motion, it is found to be about 1.6 times as large as the said one (10).

CONCLUSIONS

Based on the discussions described above, the author clarifies the principle of earthquake resistant design of gravity type dam and proposes, as the conclusions, the method of earthquake resistant design.

An author's proposal for seismic coefficient for design

<u>For empty reservoir</u>

(1) Distribution of seismic coefficient: Assume a triangular distribution along the height, and a sine type distribution along the length (containing the corrected ones) (See Fig. 6).

(2) Absolute value of seismic coefficient: Assume the area of distribution of seismic coefficient as the value "seismic coefficient of ground times height of dam" multiplied by β. And determine the seismic coefficient of ground by considering the regional degree of danger of earthquake (25), the effects of ground foundation, and the effects of dam failure upon the downstream area.

<u>For full reservoir</u>

(1) Distribution of seismic coefficient: Same as for empty reservoir.

(2) Absolute value of seismic coefficient: Assume the seismic force working on dam body as the same as for empty reservoir, and use the corrected Westergaard's approximate eq. (12) for hydrodynamic pressure.

Allowable stresses

Determine the allowable stresses for compression, tension and shear by dividing by proper factors of safety the values obtained by the standard strength tests.

Namely, the author proposes to take the actually working load as the external load at the time of earthquake occurrence, and to adopt the smaller value of safety factors than those adopted now for statical loads in determining the allowable stresses.

The author's opinion is that the value of β is to be chosen as 1.2~1.8, the value of C_0 in eq. (12) as 1.2~1.6, and the factor of safety as 3~4. Even if the above method is used, there remains the most important problem namely how the value of seismic coefficient of ground motion is to be assumed. Further studies on this problem are expected to be developed, and now it seems that there are no data except the fact that the dams designed by the conventional seismic coefficient (26) have been safe for the past strong earthquakes.

In ending this paper, author wish to add the following information for reference. When the stresses in a dam are calculated for cases of the vertical uniform ($k_c = 0.12$) and of the equivalent triangular distributions of seismic coefficient for design, small values of tensile stresses occur in upstream surface for empty reservoir and in downstream surface for full one, respectively. (16).

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NOMENCLATURE

A : Acceleration spectrum, one-dimensional.

A' : Ditto, two-dimensional.

b : Base width of dam.

C : Constant depending upon the normal mode, one-dimensional.

C' : Ditto, two-dimensional.

C_{BS} : Coefficient of bending-shear vibration, (constant determined by the form of dam section $\alpha = b/h$ and Poisson's ratio σ of dam material).

C_B : Coefficient of bending vibration, (constant determined by the α).

C_S : Coefficient of shear vibration, (constant determined by Poisson's ratio σ).

C_o : Constant to be determined by the ratio of characteristics (period, wave form, duration time) of ground motion to the natural period of dam.

C_m : Zanger's coefficient of the inclined wall.

E : Young's modulus of dam material.

g : Acceleration of gravity.

G : Shear modulus of elasticity of dam material.

h : Dam height, water depth or maximum water depth.

h' : Mean water depth.

K : Seismic coefficient, one-dimensional.

K' : Ditto, two-dimensional.

K_r : Ditto, with rocking motion.

k_e : Ditto, for the hydrodynamic pressure.

k_e' : Ditto, ditto, at the top of wall.

n : Natural circular frequancy of dam without damping, one-dimensional.

n' : Ditto, two-dimensional.

n_r : Ditto, with rocking oscillation.

p : Westergaard's approximate hydrodynamic pressure.

p' : Hydrodynamic pressure in the case of rigid wall caused by rocking oscillation.

p_s : Transient hydrodynamic pressure, caused by parallel and rocking oscillation.

T : Natural period of dam, $T=2\pi/n$.

T_r : Ditto, $T_r=2\pi/n_r$, with rocking motion.

T_B : Ditto, in the case of bending vibration.

T_{BS} : Ditto, in the case of bending-shear vibration.

T_S : Ditto, in the case of shear vibration.

T_w : Fundamental resonant period of hydrodynamic pressure of the rectangular form of exposed water area.

T_w' : Ditto, of the any form of exposed water area.

w : Weight of unit volume of water.

X : Fundamental normal mode of dam, in the direction of the length.

Y : Ditto, in the direction of the height.

α : Ratio of the base width of dam b to height h, $\alpha=b/h$.

$\alpha(t)$: Acceleration of earthquake motion.

δ : Deflection of the point where unit load acts on mass, in the case where the foundation is fixed.

η : Ditto, in the case where the foundation can only rotate.

ε : Damping coefficient of dam (ratio to critical damping), one-dimensional.

ε' : Ditto, two-dimensional.

ρ : Density of dam material.

σ : Poisson's ratio.

ω : Natural circular frequency of dam with damping, one-dimensional.

ω' : Ditto, two-dimensional.

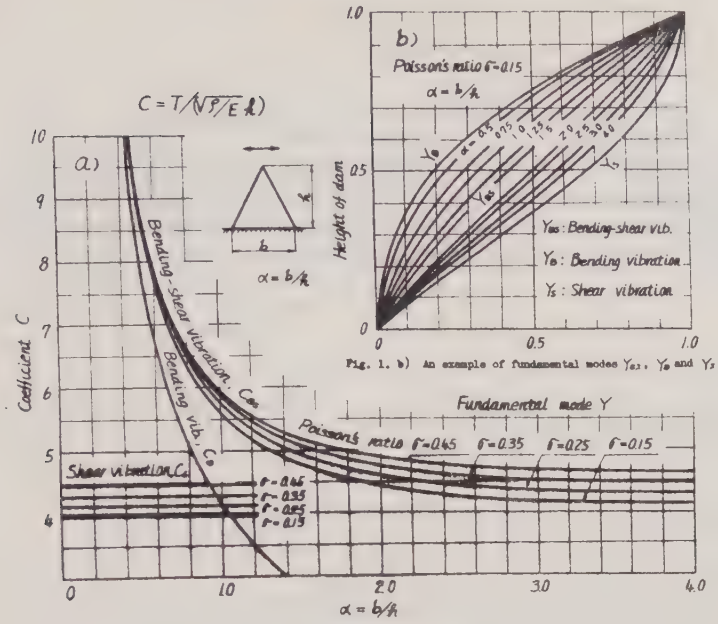

Fig. 1. b) An example of fundamental modes Y_{as}, Y_b and Y_s.

Fig. 1 a) Coefficients C_{as}, C_b and C_s.

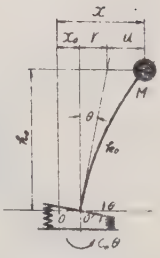

Fig. 2
The model of dam with rocking motion

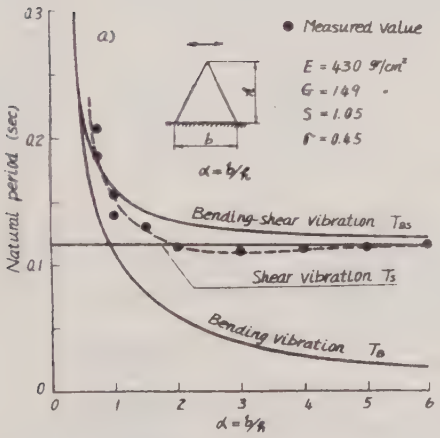

Fig. 3 a)
Relation between the calculated periods T_{as}, T_b, T_s and the measured one, (agar-agar model).

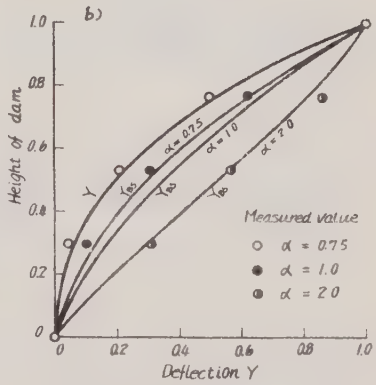

Fig. 3 b)
Relation between the calculated modes Y_{as}, Y_b, Y_s and the measured measured one, (agar-agar model).

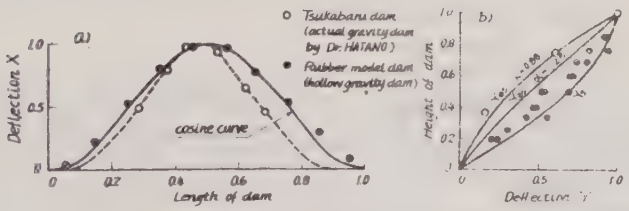

Fig. 4 Relation between the calculated modes and the measured one. Tsukabaru dam (actual gravity dam), and the rubber model dam (hollow gravity dam).

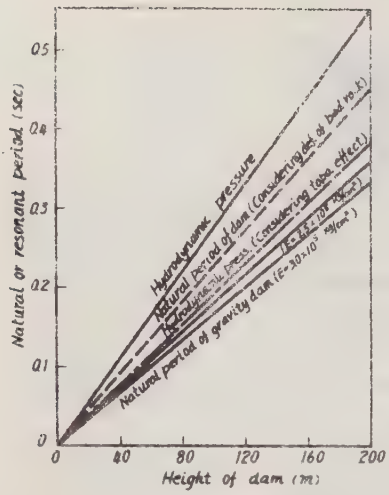
Natural or resonant period (sec)
Height of dam (m)
Hydrodynamic pressure
Natural period of dam (Considering effect of hydrodynamic pressure)
Natural period of dam (Considering effect of hydrodynamic pressure)
Natural period of gravity dam (E=3.0×10⁵ kg/cm²)

Fig. 5
Relation between the natural period of gravity dam and its height, (calculated values).

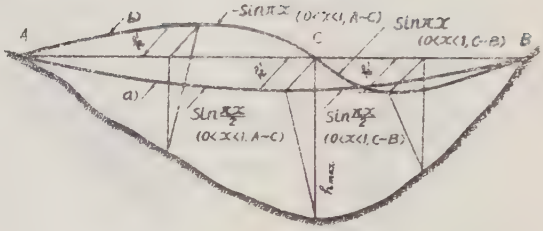

Fig. 6 Seismic coefficient for design.

Table 1. Natural periods of the hollow gravity dam models.

Height of dam element (cm)	Reservoir empty				Reservoir is full	
	Measured period (sec)			Calculated period Tes (element)	Measured period (sec) (dam)	Calculated period (sec) (element)
	Element	Dam	Dam (one body)			
	①	②	③	④	⑤	⑥
I 36.0	0.039~40	1st resonance 0.035	1st resonance 0.032	0.0394	1st resonance 0.040~46	0.0466
II 33.0	0.034~45	2nd resonance 0.026		0.0361	2nd resonance 0.033~38	0.0427
III 23.5	0.026~27			0.0257		0.0300
IV 16.0				0.0175	3rd resonance 0.030~32	0.0204

Table 2. Natural period of the gavity dam, (ploto type).

Name of dam	Height (m)	$\alpha = b/h$	Calcurated period(sec) $E=2.5\times10^5$ (Kg/cm²)	Calcurated period(sec) $E=3.0\times10^5$ (Kg/cm²)	Measured period (sec)	Vibrating force	References
Taifu	50.9	0.90	0.094	0.085	0.6 ~ 0.7 0.35 ~ 0.4	Natural earthquake over flow	9)
Miura	84.1	0.91~0.98	0.155	0.141	0.6 , 1.2 0.2 , 0.6	Blasting (Reservoir empty) Blasting (Reservoir is full)	9)
Tsukabaru	80.0	0.88	0.147	0.134	0.146	Oscillator (Reservoir is full)	18)
Hikihara	66.0	0.86	0.122	0.111	0.13	Natural earthquake (Reservoir is full)	19)

Table 3. Damping coefficient of dam, (ploto type).

Name of dam	Type of dam	Height (m)	Damping coefficient ratio to Critical		References
			Reservoir empty	Reservoir is full	
Naramata (Coffer dam)	Arch	12.8	0.08 0.08	0.10 0.15	20) 21)
Tsukabaru	Gravity	80.0	—	0.10 ~ 0.12	18)
Tonoyama	Arch	62.0	—	0.04	22)

DISCUSSION

S. Okamoto, University of Tokyo, Japan:

In order to establish a rational aseismic design procedure for dams, I think that, it is a most important problem to obtain the more complete informations about the properties of earthquake motion in mountain district. How do you think about it?

M. Hatanaka:

From the view-point of achieving a more reasonable, economic, aseismic design of dams, I believe that it is very significant to obtain seismograph records of earthquake motion at the dam site as well as the motion of a dam itself.

An observation of earthquake ground motion and the response of gravity dams is being carried out by Dr. Hatano of the Central Research Institute of Electric Power Industry, Tokyo, and also I have been making a similar observation since 1957 at Hikihara Dam in Hyogo Prefecture, Japan. The Hikihara Dam is a gravity dam, 66 meters in height, and is now equipped with a total of 6 seismometers. Namely, each pair of accelerographs and displacement seismometers is located at the top and bottom of the central part of the dam, respectively, and two other displacement seismometers are set on the both sides of the dam.

However, only a few records of the small-size earthquakes were obtained so far in the observation, probably because the area has experienced very few earthquakes.

Prof. Raphael stated a couple of hours ago that such observations are not carried out in the United States, but I hope this kind of observations will herewith be made in every earthquake countries in the world.

R. Tanabashi, Kyoto University, Japan:

1. For large-scale structures such as dams, it is often noticed that a certain difference in the phase, or phase angle, of earthquake ground motions may be observed at the boundaries of the structures, namely that vibration of the structure at its one end will be different in phase from that at the other end. Is there any considerations on the fact of phase difference, which are reflected currently in the aseismic design of dams?

2. Although it is of great significance that people at Kobe University are making an observation of response of gravity dams due to small-scale earthquakes, I realize that such an observation or study is not enough if this is carried out by only a university or an institute, and that a more comprehensive study on the problem in a larger scale, by organizing and in coopera-

tion of a number of institutions, will be indispensable. It seems to me that the government should help for these important investigations so as to maintain them with an expectation of early success. On the occasion of the 2nd World Conference on Earthquake Engineering, I strongly would like to emphasize this point.

M. Hatanaka:

1. As to the first part of Question 1, I am in complete agreement with Prof. Tanabashi's opinion. It is true that in case of high dams the phase difference between the earthquake responses of the both sides of a dam is a problem.

In general, the foundation of concrete dams is usually on a solid layer of rock, for which it is said that the longitudinal wave of earthquake travels at a speed of about 3,000 to 5,000 m/sec. If the value of Poisson's ratio of the rock layer is assumed to be $\frac{1}{4}$, the velocities of traveling transverse and Rayleigh waves will be in the range of about 1,700 to 2,900 m/sec and 1,600 to 2,700 m/sec, respectively. On the other hand, the predominant period of earthquake ground motions observed so far ranges about 0.1 to 0.3 sec. Therefore, the half wave length of the longitudinal, transverse and Rayleigh wave components of seismic waves will be about 150 to 750 meters, 85 to 435 meters, and 80 to 405 meters for each of them written in that order.

Consequently, it is easily seen that a dam with the top span length of such a scale may possibly have a relative displacement between the both sides of the dam, in the direction parallel to the stream.

Nevertheless, I do not think that this fact is taken into consideration in the prevalent design criteria for earthquake-resistant dams. But, this problem is of great importance especially for arch dams. And I have once made a proposal of a new distribution of the seismic factor or base shear coefficient, by laying an emphasis on the phase difference in the motions of both sides of dams subjected to earthquake excitation. (Ref. 1)

Also, in my paper presented at this Conference and now being discussed. I am making another proposal of a distribution of design coefficient for base shears, which varies in a pattern of one wave length of sinusoidal function. (see Fig. 6, b)

2. I am in favor with Dr. Tanabashi's comment. While investigations on earthquake response of building structures are carried out much in Japan, more attention should be paid to the aseismic design of dam structures. As Dr. Tanabashi mentioned, it is our hope that a comprehensive investigation on the earthquake problems of dam structures will be in progress in a nation wide scale and further in an international scale under the support of governmental organizations.

I deeply appreciate Profs. Okamoto and Tanabashi for their valuable discussions and comments.

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A CONSIDERATION ON EARTHQUAKEPROOF DESIGN METHOD OF EARTH DAM

Isao Minami

In designing a dam, we must take into consideration the force of earthquakes working against the dam body, especially in Japan earth dams often damaged by earthquake. The theoretical analysis of elastic vibration of dam body by Dr. Matsumura (1), Dr. Hatanaka (2) and Minami (3) etc. is an example of a study on the stability of earth dams in earthquake. But it requires future research to get any practical result. But distribution of seismic coefficient of the dam body was approximately ascertained theoretically.

Even today we use usually the sliding circle to calculate slope stability of earth dam in earthquake, which was naturally based on the assumption that the earth dam is in plastic equilibrium. In this report too, we have studied the stability of the dam by using the sliding circle with little reference to its elasticity. And also we have tried to get the safest slope of earth dam in examining these theoretical results reference to some experiments.

We have discovered some interesting facts concerning the study of the relationship between stability of earth dam and its type in the case of an earthquake. Especially I have treated vibration character of center core earth dam.

Judging from this consideration, center core type earth dam has smaller safety against vibrations because the upstream body of earth dam is usually saturated with water and modulus of rigidity of upstream side soils is smaller than that of downstream side soil. For this reason the period of free vibration would be longer and amplitude of dam body would increase in earthquake.

In stability analysis the author proposed the formula of the most dangerous sliding circle, and by this theory, we can calculate coefficient of stability without the trial and error method, and his theory can be graphed in all kinds of soil and dam heights.

1. Analytical Consideration of Center Core Type Earth Dam Vibration.

Analysis of vibration of earth dam had been studied by Dr. Matsumura (1), Dr. Hatanaka (2) and Minami (3) etc. on the assumption of shear vibration. But designing a dam, we should know that practical studies of earthquakeproof design method have not been done hitherto. The author proposes a practical

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theory of earthquakeproof design formula of earth dam in this paper. Seismic coefficient in earthquake is calculated by amplitude of dam body. For this reason the analysis of earth dam vibration is important. Actually the vibration of earth dam in earthquake is unharmonic and unsteady. But the assumption of harmonic vibration of dam body is important for convenience of simpler handling of earth dam, can be understood with approximate satisfaction by harmonic vibration analysis.

i) Two dimensional analysis of amplitude distribution of center core type earth dam.

I assume the species of vibration of earth dam is shearing type in general. The differential equation of shearing vibration is noted as follows. In Fig. 1 the external forces action to x direction against elementary portion ABCE are as follows:
to AB plane

$$-S = -b_1\left(G_1\frac{\partial w}{\partial y} + \mu_1\frac{\partial^2 w}{\partial y\partial t}\right) - b_2\left(G_2\frac{\partial w}{\partial y} + \mu_2\frac{\partial^2 w}{\partial y\partial t}\right) \tag{1}$$

to CD plane

$$S + \frac{dS}{dy}dy = b_1\left(G_1\frac{\partial w}{\partial y} + \mu_1\frac{\partial^2 w}{\partial y\partial t}\right) + \frac{\partial}{\partial y}\left(b_1G_1\frac{\partial w}{\partial y} + b_1\mu_1\frac{\partial^2 w}{\partial y\partial t}\right) + b_2\left(G_2\frac{\partial w}{\partial y} + \mu_2\frac{\partial^2 w}{\partial y\partial t}\right) + \frac{\partial}{\partial y}\left(b_2G_2\frac{\partial w}{\partial y} + b_2\mu_2\frac{\partial^2 w}{\partial y\partial t}\right) \tag{2}$$

Because the summation of external forces is equal to inertia force, kinematic equation of ABCD is as follows:

$$(\rho_2 b_2 + \rho_1 b_1)\frac{\partial^2 w}{\partial t^2} = \frac{\partial}{\partial y}\left(b_1G_1\frac{\partial w}{\partial y} + b_1\mu_1\frac{\partial^2 w}{\partial y\partial t}\right) + \frac{\partial}{\partial y}\left(b_2G_2\frac{\partial w}{\partial y} + b_2\mu_2\frac{\partial^2 w}{\partial y\partial t}\right) \tag{3}$$

In fundamental triangle, since b is shown by next equation

$$b_1 = m_1 y \quad , \qquad b_2 = m_2 y \, . \tag{4}$$

next equation is gained

$$\frac{\partial^2 w}{\partial t^2} = \frac{1}{(m_1\rho_1 + m_2\rho_2)y}\frac{\partial}{\partial y}\left[y(m_1G_1 + m_2G_2)\frac{\partial w}{\partial y} + y(m_1\mu_1 + m_2\mu_2)\frac{\partial^2 w}{\partial y\partial t}\right] \tag{5}$$

$$= \frac{m_1G_1 + m_2G_2}{m_1\rho_1 + m_2\rho_2}\left(\frac{\partial^2 w}{\partial y^2} + \frac{1}{y}\frac{\partial w}{\partial y}\right) + \frac{m_1\mu_1 + m_2\mu_2}{m_1\rho_1 + m_2\rho_2}\left(\frac{\partial^3 w}{\partial y^2\partial t} + \frac{1}{y}\frac{\partial^2 w}{\partial y\partial t}\right)$$

$$C_0^2 = \frac{m_1G_1 + m_2G_2}{m_1\rho_1 + m_2\rho_2} \quad , \qquad C_1^2 = \frac{m_1\mu_1 + m_2\mu_2}{m_1\rho_1 + m_2\rho_2} \tag{6}$$

the differential equation of shear vibration

$$\frac{\partial^2 w}{\partial t^2} = C_0^2\left(\frac{\partial^2 w}{\partial y^2} + \frac{1}{y}\frac{\partial w}{\partial y}\right) + C_1^2\left(\frac{\partial^3 w}{\partial y^2\partial t} + \frac{1}{y}\frac{\partial^2 w}{\partial y\partial t}\right) \tag{7}$$

and we showed of the ground vibration

$$w_0 = d\cos\frac{2\pi}{T}t \tag{8}$$

$$w = f_{(y)}e^{ipt} \quad , \quad p = \frac{2\pi}{T} \tag{9}$$

Since, differential equation of harmonic vibration is as follows:

$$\frac{\partial^2 f}{\partial y^2} + \frac{1}{y}\frac{\partial f}{\partial y} + \frac{p^2}{C_0^2 + iC_1^2 p} = 0 \tag{10}$$

Boundary condition is

$$\begin{aligned} &y=0 \quad , \quad \partial f/\partial y = 0 \\ &y=b \quad , \quad f = d \end{aligned} \tag{11}$$

The solution of dam vibration is as follows:

$$f = d\,\frac{1}{J_0(mb)}\,J_0(my) \tag{12}$$

where

$$\begin{aligned} m &= \frac{1}{\sqrt{2}}\left[A+\sqrt{A^2+B^2}\right]^{1/2} \quad , \quad A = \frac{p^2C_0^2}{C_0^4+p^2C_1^4} \\ n &= \frac{1}{\sqrt{2}}\left[-A+\sqrt{A^2+B^2}\right]^{1/2} \quad , \quad B = \frac{1}{2}\,\frac{C_1^2p^3}{C_0^4+C_1^4p^2} \end{aligned} \tag{13}$$

Two dimensional amplitude analysis of homogeneous type earth dam is treated with above consideration in the case of $C_0^2 = G/\rho$, $C_1^2 = \mu/\rho$ instead of eq. (6).

ii) Three dimensional vibration analysis of earth dam.

Analytical formula is introduced from the differential equation of harmonic shear vibration for three directions in Fig. 3.

$$\frac{\partial^2 w}{\partial t^2} = C_0^2\left(\frac{\partial^2 w}{\partial x^2}+\frac{\partial^2 w}{\partial y^2}\right)+C_1^2\left(\frac{\partial^3 w}{\partial x^2\partial t}+\frac{\partial^3 w}{\partial y^2\partial t}\right)+C_0^2\frac{1}{y}\frac{\partial w}{\partial y}+C_1^2\frac{1}{y}\frac{\partial^2 w}{\partial y\partial t} \tag{14}$$

When the base ground has vibration of $d\sin pt$, the deformation of some point of earth dam is shown in the following formula.

Where boundary condition is as follows:

$$\left.\begin{aligned} &x=0 \ , \quad x=a \ , \quad w = d\sin pt \\ &y=0 \ , \quad \frac{\partial w}{\partial y}=0 \ , \quad y=b \ , \quad w = d\sin pt \end{aligned}\right\} \tag{15}$$

When we analyse eq. (14) from the boundary condition eq. (15).

$$w = d\Bigg[\sum_{s=1}^{\infty}\frac{2}{b^2\left[J_1(\lambda_s)\right]^2}\Bigg\{\cos m_s x\cosh n_s x+\frac{(\cosh n_s a-\cos m_s a)(\sin m_s a\sin m_s x\sinh n_s x+\sinh n_s x\sinh n_s a\cos m_s x)}{\sin^2 m_s a\cosh^2 n_s a+\sinh^2 n_s a\cos^2 m_s a}\Bigg\}J_0\left(\frac{\lambda_s}{b}y\right)\lambda\,d\lambda \tag{16}$$

$$-\frac{4}{\pi}\,\frac{\sin\frac{n_s\pi}{a}x\left\{J_0(m_s y)J_0(n_s y)+2(-J_2(m_s y)J_2(n_s y)+\cdots\cdots\cdots)\right\}}{J_0(m_s b)J_0(n_s b)+2\left\{-J_2(m_s b)J_2(n_s b)+J_4(m_s b)J_4(n_s b)+\cdots\right\}}\Bigg]$$

where

$$\left.\begin{aligned} m_s &= \frac{1}{\sqrt{2}}\left(A_s+\sqrt{A_s+B_s}\right)^{1/2}, & A_s &= \frac{p^2C_0^2}{C_0^4+p^2C_1^4}-\left(\frac{\lambda_s}{b}\right)^2 \\ n_s &= \frac{1}{\sqrt{2}}\left(-A_s+\sqrt{A_s+B_s}\right)^{1/2}, & B_s &= \frac{C_1^2p^3}{C_0^4+p^2C_1^4} \\ m_s &= \frac{1}{\sqrt{2}}\left(A_s+\sqrt{A_s+B_s}\right)^{1/2}, & A_s &= \frac{p^2C_0^2}{C_0^4+p^2C_1^4}-\left(\frac{n\pi}{a}\right)^2 \\ n_s &= \frac{1}{\sqrt{2}}\left(-A_s+\sqrt{A_s+B_s}\right)^{1/2}, & B_s &= \frac{C_1^2p^3}{C_0^4+p^2C_1^4} \end{aligned}\right\} \tag{17}$$

2. Distribution of Seismic Coefficient.

The accerelation acting on the some point of earth dam is calculated by next equation in harmonic vibration.

$$\alpha' = -\frac{\partial^2 w}{\partial t^2} \quad , \quad k = \alpha'/g \tag{18}$$

Since w are given eq. (12)

$$\alpha' = -p^2 \frac{d}{J_{o(mb)}} J_{o(my)} \sin pt \tag{19}$$

and maximum accerelation is at $\sin pt = 1$.

$$\alpha'_{max} = -p^2 \frac{d}{J_{o(mb)}} J_{o(my)} \tag{20}$$

Fig. 3 shows the distribution of α'/g . But α'/g distribution must be calculated by eq. (20). The author proposed next assumption for practical design.

Now at crest

$$\alpha'/g = -\frac{p^2 d}{g J_{o(mb)}} \tag{21}$$

at the base of earth dam

$$\alpha'/g = -p^2 d/g = k_o$$

and specific amplitude is $\frac{w}{w_o} = \frac{J_{o(my)}}{J_{o(mb)}}$.

3. Experimental Consideration.

The vibration box is used for an experiment to clarify earth dam vibration problem. The property of vibration box has 0.8 meter width, 1.8 meter length and 0.5 meter depth, and available period of vibration is in the range of 0.05----0.18 second. Height of model is 30 cm, and acceration was messured on the dam body. At the crest the several fold amplitude of base vibration was seen. At elastic vibration, eq. (20) is available. On calculation of α , the value of modulus of rigidity G and coefficient of viscosity are not clarified in Otani Earth Dam (N. B. paragraph 6). But G-value is given in general as 50 kg/cm^2--280 kg/cm^2, here G = 50 kg/cm^2 is assumed by the experiment, but coefficient of viscosity is not clear. In the case where the amplitude of the model is small, the eq. (20) shows good approximation. But at larger amplitude, the coefficient of viscosity would be increased. The rupture of earth dam is limited to certain places of dam but most parts of dam are still safe. I used the vibration analysis eq. (20) approximately. Shape of sliding surface is varied according to slope angle of dam, shearing strength of soil (cohesive soil or noncohesive soil) and type of earth dam.

i) Effect of slope angle on the shape of sliding surface:

There are two mechanical types of rupture by earthquake, and it due to slope angle of dam. In authors experiment at the slope angle 45°, the tension crack was found at first on the slope. But at the slope angle 35° it could not find tension crack but sliding crack at first near the dam crest. Of course

these phenomena depend on also mechanical property of soil.

For that reason, we assumed that the fundamental equation of earth dam vibration is approximately that of shear vibration types.

ii) Effect of shearing strength of soil.

According to my theoretical consideration, in noncohesive soil the depth perpendiculer to slope surface of sliding circle is zero it is proved by the experiment, this property was found and sliding line was straight. In cohesive soil, according to my theoretical consideration and experiments, the sliding surface would be circular type. Using these theories as a basis, the author deduce the analytical formula of earth dam slope by sliding circles, which is as follows.

iii) Effect of shape of the earth dam.

For the present I classify the shape of earth dam into the following three types.

(i) homogeneous type,
(ii) center core type,
(iii) type with impervious board on the upstream slope surface.

In types (i) and (ii), the sliding surface was circular in cohesive soil and straight in noncohesive soil, and upstream slope was weaker than the downstream slope in earthquake when fully reservoired. In type (iii), the definite type of sliding had not been found. In many experiments, upstream slope was stronger than the down stream slope. The author treated in this paper the stability of slope in types (i) and (ii) against vibration.

5. Stability Analysis of Earth Dam Slope.

Sliding circle method is one of the most practical for designing of earth dam slope. But because Taylor, Fellenius and Patterson's sliding circle method, hitherto used, and it requires trial and error method for calculation and much labour must be put into safety analysis of slope. Especially when the seismic coefficient is assumed in eq. (20), trial and error method requires greater time and labour consumption.

To save this labour, the author deduced a new method, it follows.

In this method, the conception of the most dangerous sliding circle is used, and most dangerous sliding circle can be calculated in only one step (without trial and error method).

In Fig. 4 area of sliding body ABED is as follows;

$$F = R^2(\varphi - \cos\varphi \sin\varphi) \tag{22}$$

From geometrical property of sliding circle next formula is gained.

$$\frac{H}{\sin\alpha} = 2R\sin\varphi$$
$$h = R(1 - \cos\varphi) \tag{23}$$

If we neglect the φ from eq. (22) and eq. (23)

$$R = \frac{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2}{8h} \tag{24}$$

and radius of sliding circle can be shown by h only.

From eq. (23) φ is shown as follows;

$$\varphi = \sin^{-1} \frac{4h\left(\frac{H}{\sin\alpha}\right)}{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2} \tag{25}$$

In Fig. 4 the following relation is seen generally.

$$h \ll \frac{H}{\sin\alpha} = l \tag{26}$$

Eq. (25) could develope convergent series as follows;

$$\varphi = \frac{4h\frac{H}{\sin\alpha}}{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2} + \frac{1}{2\cdot 3}\left\{\frac{4h\left(\frac{H}{\sin\alpha}\right)}{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2}\right\}^3 + \frac{1\cdot 3}{2\cdot 4\cdot 5}\left\{\quad\right\}^5 + \cdots\cdots \tag{27}$$

and

$$F \fallingdotseq \left\{\frac{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2}{8h}\right\}^2 \left\{\frac{4h\left(\frac{H}{\sin\alpha}\right)}{4h^2 + \left(\frac{H}{\sin\alpha}\right)^2} - \frac{4h\frac{H}{\sin\alpha}\left(\frac{H^2}{\sin^2\alpha} - 4h^2\right)}{\left(4h^2 + \frac{H^2}{\sin^2\alpha}\right)}\right\} \tag{28}$$

The angle between resultant force line and vertical axis θ is shown as follows:

$$\theta = \tan^{-1} \frac{k_s}{1 \pm k_v} \fallingdotseq \tan^{-1} k_s \tag{29}$$

own weight of sliding body is

$$W = \rho F \tag{30}$$

The length between center of sliding circle and center of gravity Q.

$$OQ = \frac{4}{3} \frac{R\sin^3\varphi}{2\varphi - \sin 2\varphi} = \frac{H^3}{12 F \sin^3\alpha} \tag{31}$$

moment of sliding out

$$\frac{M_-}{1+k^2} = W \cdot OQ \cdot \sin(\alpha+\theta) = \frac{\rho H^3}{12} \frac{\sin(\alpha+\theta)}{\sin^3\alpha} \tag{32}$$

Resistant moment by cohesive force C

$$M_c = 2R^2\varphi C \tag{33}$$

Resistant moment by internal friction angle ϕ

$$\frac{M_f}{1+k^2} = W\cos(\alpha+\theta-d\theta)\tan\phi R \tag{34}$$

Small angle $d\theta$ between OD line and OE lines

$$d\theta = \frac{ds}{R} \ll 1 \tag{35}$$

and

$$\cos\left(\alpha+\theta-\frac{ds}{R}\right) = \cos(\alpha+\theta)\cos\frac{ds}{R} + \sin(\alpha+\theta)\sin\frac{ds}{R} \tag{36}$$

From geometrical property of Fig. 4.

$$\frac{R}{\sin(\pi-\theta-\alpha)} = \frac{QE}{\sin d\theta} = \frac{OQ}{\sin(\alpha+\theta-d\theta)} \tag{37}$$

From eq. (37)

$$\frac{OQ}{\sin(\alpha+\theta-d\theta)} = \frac{R}{\sin(\pi-\theta-\alpha)} \tag{38}$$

$$\sin(\alpha+\theta-d\theta) = \frac{OQ}{R}\sin(\pi-\theta-\alpha) = \sin(\alpha+\theta)\cos d\theta - \sin d\theta\cos(\alpha+\theta) \tag{39}$$

$$\cos\frac{ds}{R} = 1 - \frac{1}{2}\left(\frac{ds}{R}\right)^2 + \cdots\cdots \doteqdot 1 \tag{40}$$

$$\sin d\theta = \tan(\alpha+\theta) - \frac{H^3}{12\sin^3\alpha}\,\frac{\sin(\pi-\theta-\alpha)}{\cos(\alpha+\theta)}\,\frac{1}{FR} \tag{41}$$

$$\cos(\alpha+\theta-d\theta) = \left\{\cos(\alpha+\theta) + \sin(\alpha+\theta)\tan(\alpha+\theta) - \frac{H^3}{12\sin^3\alpha}\sin(\pi-\alpha-\theta) \times \tan(\alpha+\theta)\,\frac{1}{FR}\right\} \tag{42}$$

where

$$\frac{Mf}{1+k^2} = \left\{\cos(\alpha+\theta) + \sin(\alpha+\theta)\tan(\alpha+\theta)\right\}\rho\tan\phi\,FR - \frac{H^3}{12\sin^3\alpha}\sin(\pi-\alpha-\theta)\tan(\alpha+\theta)\,\rho\tan\phi \tag{43}$$

Total resistant moment

$$M_- = M_c + M_f$$

Safety coefficient against sliding

$$\eta = \frac{M_f + M_c}{M_-} \tag{44}$$

Position of most dangerous slip circle on earth dam slope is calculated by function h, the depth of sliding circle, and h can be get followings

$$\frac{\partial\eta}{\partial h} = 0 \tag{45}$$

where

$$\frac{\partial\eta}{\partial h} = \frac{1}{M_-}\frac{\partial}{\partial h}(M_f+M_c) = \frac{1}{M_-}\left[\frac{\partial M_f}{\partial h} + \frac{\partial M_c}{\partial h}\right] \tag{46}$$

where

$$\frac{\partial\eta}{\partial h} = \frac{H}{2\sin\alpha\,h^2}\left[\left\{\cos(\alpha+\theta) + \sin(\alpha+\theta)\tan(\alpha+\theta)\right\}(1+k^2)\tan\phi\rho h^3 - ch^2 - \frac{cH^3}{4\sin^2\alpha}\right] \tag{47}$$

Now h_m is led by next formula

$$\{\cos(\alpha+\theta)+\sin(\alpha+\theta)\tan(\alpha+\theta)\}(1+k^2)\tan\phi\,\rho h_m^3+ch_m^2-\frac{cH^2}{4\sin^2\alpha}=0 \tag{48}$$

where we set as follows: and depth of most dangerous sliding circle is calculated by next formula

$$h_m=\frac{1}{3z_m}(a-3z_m^2)-\frac{a'}{3} \tag{49}$$

Now I got the depth of sliding circle from the slope surface. Thus I could gain the most dangerous safety factor by next equation.

$$\eta_{min.}=\frac{M_f(h_m)+M_c(h_m)}{M_-} \tag{50}$$

where

$$\frac{H}{\sin\alpha}=l\ ,\quad \frac{c}{\rho\tan\phi}=\kappa'\ \text{ and }\ \frac{4h_m l}{4h_m^2+l^2}=\phi \qquad d=\frac{l^2}{4}a' \qquad a=-\frac{a'^2}{3}, \tag{51}$$

$$\frac{M_f}{1+k^2}=\{\cos(\alpha+\theta)+\sin(\alpha+\theta)\tan(\alpha+\theta)\}\rho\tan\phi \times\{R\}^3\{\sin^{-1}\phi-\phi\sqrt{1-\phi^2}\}-\frac{H^3}{12\sin^3\alpha}\sin(\pi-\alpha-\theta)\tan(\alpha+\theta)\rho\tan\phi \tag{52}$$

$$M_c=2c(R^2)\sin^{-1}\phi \tag{53}$$

$$M_-=\frac{\rho}{12}l^3\sin(\alpha+\theta) \tag{54}$$

$$a'=\frac{\kappa'}{4\{\cos(\alpha+\theta)+\sin(\alpha+\theta)\tan(\alpha+\theta)\}} \tag{55}$$

$$b'=a'l^2 \tag{56}$$

$$\theta=\tan^{-1}k \tag{57}$$

$$z_m=\left[\left(\frac{a'}{3}\right)^3-\frac{d}{2}\pm\sqrt{\left\{\left(\frac{a'}{3}\right)^3-\frac{d}{2}\right\}^2-\left(\frac{a'^2}{3}\right)^3}\ \right]^{1/3} \tag{58}$$

6. A Research on Otani Earth Dam.

Otani Earth Dam is in Komatsucho Ehime Prefecture in Japan. In great earthquake of Nankaido in 1946, the dam body was destroyed as in Fig. 5. The scale of Otani Earth Dam is showed in table 1. And mechanical property of soil is as in table 2.

At the time of the Nankaido earthquake, the scale of earthquake is as in table 3.

The analytical results of earth dam slope were as follows:

i) Vibration analysis of Otani Earth Dam:

Because the data of soil mechanics of table 2 are gained by labolatory test, the analytical results have only approximate values. In eq. (19) the cohesive coefficient is not clear. But I assumed several values of coefficient. The result of calculation is as Fig. 3. As you see from Fig. 3, the earth dam undergoes more vibration than the foundation.

ii) Stability analysis of earth dam slope:

In no quaking condition, the safety coefficient of slope by eq. (50) was 1.5 at upstream side, and 2.3 at downstream side. The result of this calculation shows that in steady condition the slope is satisfactly safe. And I calculate the neccesary degree of vibration when the safety coefficient η is equal to 1. By this calculation, I presume the action of earthquake force to earth dam at total body of upstream (cracked side).

The relation between the stability factor and degree of vibration at Otani Earth Dam is as table 4.

iii) Calculation of coefficient of viscosity by presumed seismic coefficient.

Amplitude of vibration of dam body decreases in consequence of μ value. For this reason coefficient of viscosity of Otani Earth Dam could be calculated by the use of eq. (19).

The result of calculation was as table 4. From table 4, I got the coefficient of viscosity of Otani Earth Dam which was as follows:

$$\sqrt{\mu/\rho} \fallingdotseq 25 \ (\mathrm{m/sec^{\frac{1}{2}}})$$

Bibliographies;

(1) M. Matsumura : Deformation of the Earth Dams due to Earthquake Motion, Report of the Public Works Res. Inst., Ministry of Construction, No. 28, 1934, (in Japan).

(2) M. Hatanaka : Fundamental Considerations on the Earthquake Resistant Properties of the Earth Dam, Disaster Preventation Research Institute Bulletin No. 11, December, 1955.

(3) I. Minami : A Shape of Earthquakeproof Cross Section of Earth Dam, Second Discussion on Earthquake Engineering, 1959. (in Japan).

(4) Geophysical Review, No. 568, December 1946, Published by the Central Meteorological Observatory.

Symbols;

w_e : deformation of foundation in earthquake (horizontal),
w : horizontal deformation of dam by shearing vibration,
w_c : horizontal deformation of dam crest,
G : modulus of rigidity of soil (average),
ρ : unit weight of soil (average),
T : period of vibration in earthquake,
T_0 : period of free vibration of earth dam,
b : height of earth dam,
d : amplitude of foundation in earthquake,
μ : coefficient of viscosity against shearing strain (average),
t : time,
x, y, z : coordinate,
m_1, m_2 : cotangent of slope angle to vertical axis, suffix 1 is upstream and suffix 2 is down stream ($m_1 = \tan\alpha_1$, $m_2 = \tan\alpha_2$),
i : imaginary number,
g : accerelation of gravity,
α' : accerelation on dam body in earthquake,
J_0 : Bessel function,
Q : center of gravity of sliding body,
l : length of surface of the sliding surface,
h : depth from slope surface to bottom of sliding circle,
α : angle of slope against horizontal plane,
φ : angle between OA line and OQ line,
F : area of sliding body,
R : radius of sliding circle,
θ : angle between resultant force line and vertical axis,
k : seismic coefficient at dam body,
M_- : moment of sliding out,
M_f : resistant moment by internal friction,
M_c ; resistant moment by cohesion,
c : cohesion of soil,
ϕ : internal friction angle,
ds : arc length of ED,
b_1, b_2 : width of element in Fig. 1 (1; upstream, 2; down stream).
G_1, G_2 : modulus of rigidity of soil (1; upstream, 2; down stream).
μ_1, μ_2 : coefficient of viscosity of soil (1; ups. 2; dow.).
ρ_1, ρ_2 : unit weight of soil (1; ups. 2; dow.).
p : $2\pi/T$,
e : base of natural logarithm,
b ; longitudinal length of earth dam,
k_h : horizontal seismic coefficient,
k_v : vertical seismic coefficient,
W : weight of sliding body ($= \rho F$),
$d\theta$: angle between OD and OE,
η : coefficient of slope stability,
S : shearing force.

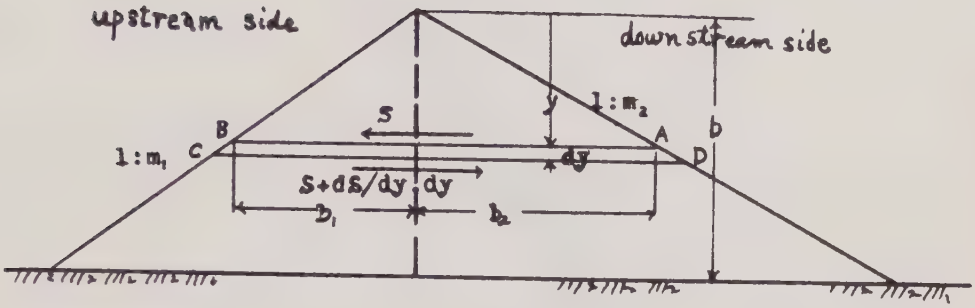

Fig. 1 : Cross section of center core type earth dam.

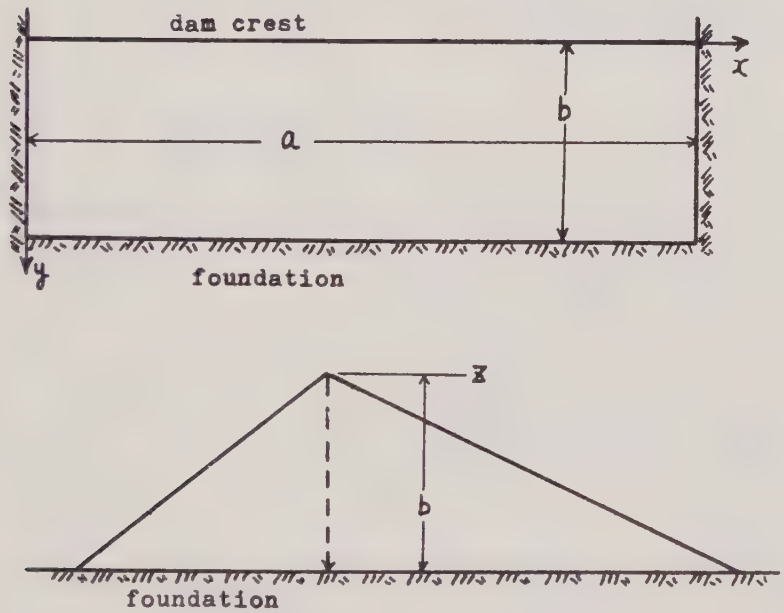

Fig. 2 : Model of 3-dimensional center core type earth dam.

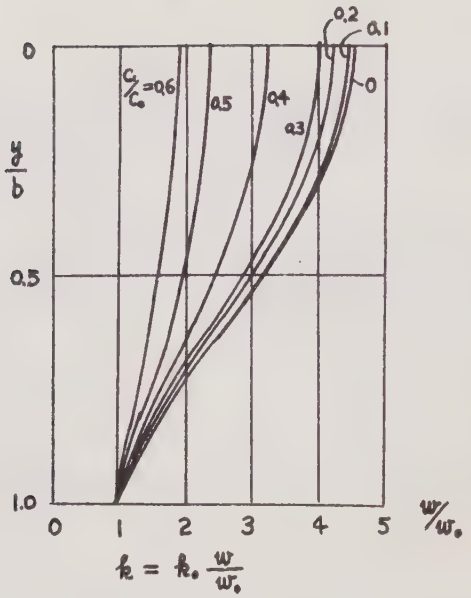

Fig. 3 : Specific amplitude distribution of earth dam and its practical assumptions.

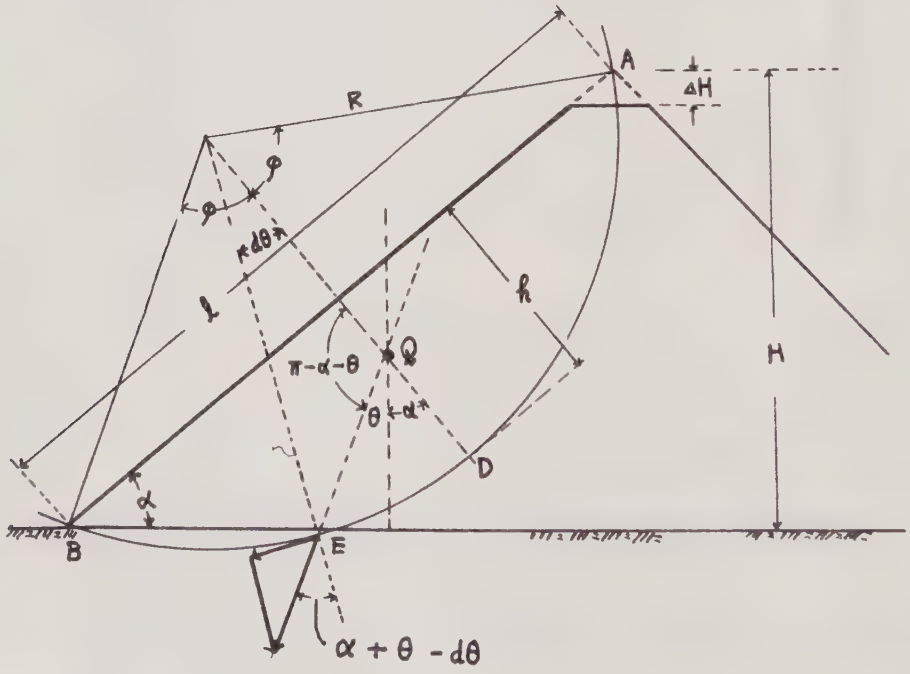

Fig. 4 : Model of circular sliding circle of the earth dam slope.

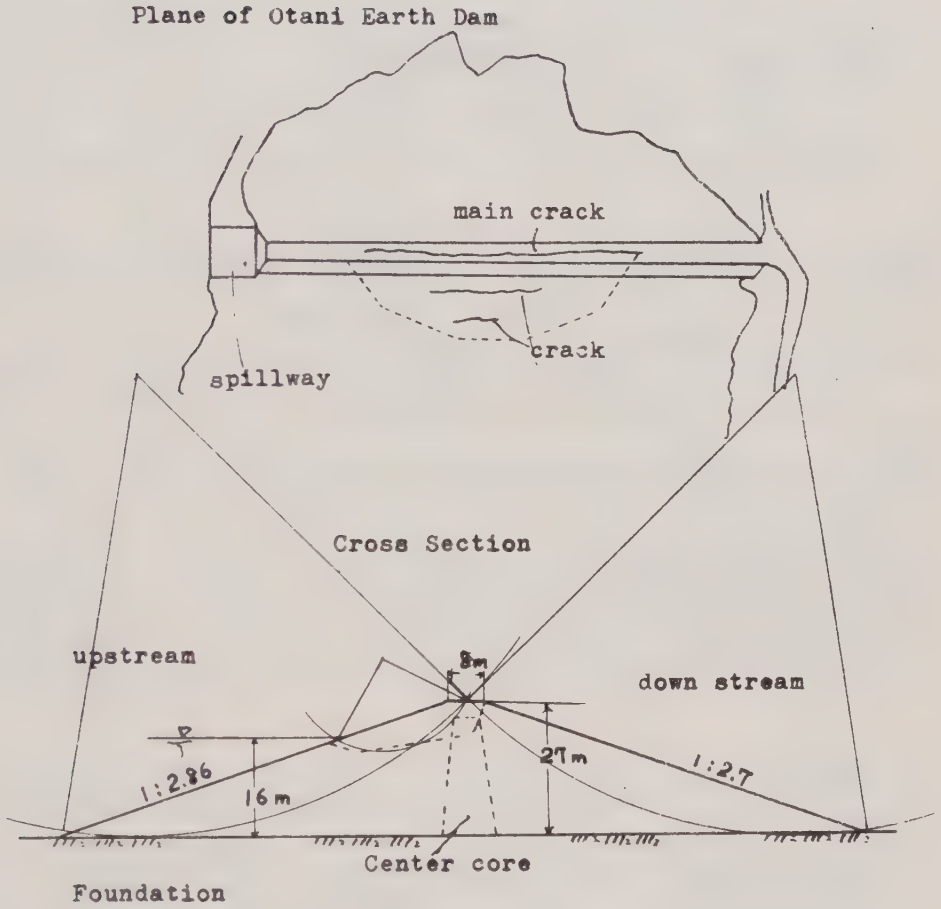

Fig. 5 : Otani Earth Dam.

Table 1 : About the scales of Otani Earth Dam,

height	slope angle upstream	down stream	water depth at the time of rupture
27 m	1: 2.86	1: 2.7	16 m

Table 2 : Mechanical properties of Otani Earth Dam's soil,

unit weight up.	dow.	ton/m	cohesion up.	dow.	ton/m	C	m/s
1.8	1.75		0.5	1.0		50	

Table 3 : Characteristic of the Nankaido Earthquake in Komatsu-cho,

degree of vibration		period of earthquake vibration		amplitude
4	(4)	2.3	Sec.	———

Table 4 : Stability coefficients of Otani Earth Dam's slope against various degree of seismic coefficients.

(up.)

seismic coefficient	k = 0	0.1	0.2	0.3
stability factor	1.5	1.1	0.9	——

(dow.)

seismic coeff.	k = 0	0.1	0.2	0.3
stability fact.	2.3	1.75	1.4	1.1

(sliding body)

seismic coeff.	k = 0.2	0.25
stability fact.	1.1	1.0

STUDY ON ARRANGEMENTS OF ASEISMATIC ELEMENTS

By SUKENOBU TANI*

1. Introduction.

This paper is the report on stiffness and lateral force distribution in the frame with aseismatic elements (bracings or walls). It has been well known that aseismatic bracings and walls to arrange in the frame are very effective against earthquakes. However, we have few studied the subjects on the reasonable arrangement of aseismatic elements. Aseismatic elements to be arrange continuously to a vertical direction have been recomended customarily. But, this arrangement has some weak points as follow; Structural planning is restricted, and in addition to, lateral force distribution of aseismatic elements decreases at the higher portion of the structure. Sometimes, contrary effect come out.

This paper showes that dispersive arrangements of aseismatic elements at each story increase freedom of planning and make uniform lateral force distribution at each story, so that we may rationalize the arrangement of aseismatic elements.

In the first, we will discuss theoretically stiffness and lateral force distribution on the continuous arrangement which have been adopted customarily, and in the next, we will discuss those of the dispersive arrangements and indicate the results of the experiment used the steel model. Lastly, we will discuss the character of the arbitrary arrangements by means of Stress-Controlled Calculation and compare the above results with the experiment used the achlilite model.

2. Stiffness and Lateral Force Distribution on Continuous Arrangement of Aseismatic Elements.

Stiffness and lateral force distribution on the continuous arrangement in elastic range will be required by finite differences and be discussed.

(2.1) Stiffness and Lateral Force Distribution on Continuous Arrangement of Bracings.

Consider the structure that is the symmetric plan, as shown in Fig 1. It is assumed that the each of frames is connected with rigid floors at each story. The elevation of the frame is shown in Fig 2. In A and D frame, bracings are arranged continuously, and the other hand, B and C frame are open frames. The assumption is that the axial force acting on the pairs of column in A(D) frame is equal to

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each other in the absolute value and contrary in the sign.

When the angle of deflection of column caused by the bending deformation at the n-th story is denoted by ${}_mR_{An}$ and that of beam is ${}_mR'_{An}$, ${}_mR_{An}$ is equal to ${}_mR'_{An}$. The relation between the deformation and the stress at n-th story is expressed as follow,

$$ {}_mR_{An} - {}_mR_{A(n-1)} = \frac{2M_{An}h_n}{A_{An}E\ell^2} \tag{1} $$

where, M_{An} = bending moment at the n-th story in A frame
A_{An} = column area at the n-th story in A frame
E = modulus of elasticity
h_n = story height at the n-th story

If the shearing force at the n-th story in A frame is denoted by Q_{An}.

$$ \left.\begin{aligned} M_{An} &= \sum_{n}^{s} Q_{Ai}h_i \\ M_{A(n-1)} &= \sum_{n-1}^{s} Q_{Ai}h_i \end{aligned}\right\} \tag{2} $$

When h_n is the same at each story, h_n may be replaced by h.

Substituting Eqs. (2) in Eq.(1)

$$ {}_mR_{An} - {}_mR_{A(n-1)} = \frac{2h^2}{A_{An}E\ell^2} \sum_{n}^{s} Q_{An} \tag{3} $$

similarly,

$$ {}_mR_{A(n-1)} - {}_mR_{A(n-2)} = \frac{2h^2}{A_{A(n-1)}E\ell^2} \sum_{n-1}^{s} Q_A \tag{4} $$

when A_{An} is the same at each story, it is replaced by A. Subtrating Eq.(3) to Eq.(4), if (n-1) is replaced by n,

$$ {}_mR_{A(n+1)} - 2{}_mR_{An} + {}_mR_{A(n-1)} = -{}_mK_A Q_{An} \tag{5} $$

$$ \text{where, } {}_mK_A = \frac{2h^2}{AE\ell^2} \text{ (bending softness coefficient)} \tag{6} $$

When the beam is much larger than column on stiffness, the shearing deformation in A frame is,

$$ {}_sR_{An} = \frac{h^2}{4EI_A\left(6 + \frac{A_d h^3}{4I_A L}\cos^2\theta\right)} Q_{An} \tag{7} $$

where, ${}_sR_{An}$ = shearing deflection angle at the n-th story in A frame.
A_d = bracing area in A frame
I_A = moment of inertia of column in A frame
L = length of bracing
θ = inclination of bracing

denoting ${}_sK_A$ as follow,

$$ {}_sK_A = \frac{h^2}{4EI_A\left(6 + \frac{A_d h^3}{4I_A L}\cos^2\theta\right)} \quad \text{(shearing softness coefficient)} \tag{8} $$

Substituting Eq.(8) in Eq.(7)

$$ {}_sR_{An} = {}_sK_A Q_{An} \tag{7a} $$

Then, as the total deformation is the sum of the bend-

ing and the shearing deformation, the below expression is given,

$$ {}_mR_{An} + {}_sR_{An} = R_{An} \tag{9} $$

similarly, in B frame

$$ {}_mR_{Bn} + {}_sR_{Bn} = R_{Bn} \tag{10} $$

If $R_{An} = R_{Bn} = R_n$, combining Eq.(5) and Eq.(7a),

$$ R_{(n+1)} - 2R_n - R_{(n-1)} = {}_sK_A Q_{A(n+1)} - (mK_A + 2{}_sK_A) Q_{An} + {}_sK_A Q_{A(n-1)} \tag{12} $$

Since B frame is open, the deformation of B frame may be considered mainly the shearing deformation, so the following expression is given,

$$ {}_sR_{Bn} = R_{Bn} = R_n = \frac{h^2}{24EI_B} Q_{Bn} \tag{13} $$

where, I_B = moment of inertia of column in B frame.

${}_sK_B$ is denoted as follow,

$$ {}_sK_B = \frac{h^2}{24EI_B} \tag{14} $$

Substituting Eq.(14) in Eq.(13)

$$ R_n = {}_sK_B Q_{Bn} \tag{13a} $$

Combining Eq.(5) and Eq.(7a), but Eq.(15) is given by using Eq.(17)

$$ R_{(n+1)} - \gamma R_n + R_{(n-1)} = (2A-B)(S+1)P - (2A-B)Pn \tag{15} $$

where,

$$ \left.\begin{aligned} p &= \frac{2 + (mK_A + 2{}_sK_A)\frac{1}{{}_sK_B}}{1 + \frac{{}_sK_A}{{}_sK_B}} \\ A &= \frac{{}_sK_A}{1 + \frac{{}_sK_A}{{}_sK_B}}, \qquad B = \frac{mK_A + 2{}_sK_A}{1 + \frac{{}_sK_A}{{}_sK_B}} \end{aligned}\right\} \tag{16} $$

$$ Q_{An} + Q_{Bn} = Q_n = (S-n+1)P \tag{17} $$

The solution of Eq.(15) is,

$$ R_n = C_1 \sinh\varphi \cdot n + C_2 \cosh\varphi \cdot n + \frac{2A-B}{2-p}(S+1-n)P \tag{18} $$

$$ \left(\because \cosh\varphi = \frac{p}{2}\right) $$

where, C_1 and C_2 are constant to be determined by boundary conditions. In the boundary conditions that is the fixed end at the base of the 1st story and is the framed structure of the S-th story, C_1 and C_2 are given as below,

$$ \left.\begin{aligned} C_1 &= \frac{U_1(p'\cosh S\varphi - \cosh(S-1)\varphi) - U_2(\cosh 2\varphi - p\cosh\varphi)}{(\sinh 2\varphi - p'\sinh\varphi)(p'\cosh S\varphi - \cosh(S-1)\varphi) - (\cosh 2\varphi - p\cosh\varphi)(p'\sinh S\varphi - \sinh(S-1)\varphi)} \\ C_2 &= \frac{-U_1(p'\sinh S\varphi - \sinh(S-1)\varphi) + U_2(\sinh 2\varphi - p'\sinh\varphi)}{(\sinh 2\varphi - p'\sinh\varphi)(p'\cosh S\varphi - \cosh(S-1)\varphi) - (\cosh 2\varphi - p\cosh\varphi)(p'\sinh S\varphi - \sinh(S-1)\varphi)} \end{aligned}\right\} \tag{19} $$

where,

$$ \left.\begin{aligned} U_1 &= \left[(S-1)A - SB - \frac{2A-B}{2-p}\{(S-1) - \gamma S\}\right]P \\ U_2 &= \left[B - 2A - \frac{2A-B}{2-p}(p'-2)\right]P \\ p' &= \frac{1 + (mK_A + {}_sK_A)\frac{1}{{}_sK_B}}{1 + \frac{{}_sK_A}{{}_sK_B}}, \qquad B' = \frac{mK_A + {}_sK_B}{1 + \frac{{}_sK_A}{{}_sK_B}} \end{aligned}\right\} \tag{20} $$

If R_n has been required, Q_{Bn} may be obtained. Considering $Q_{An} = Q_n - Q_{Bn}$, lateral force distribution ratio on A frame against B frame may be obtained as follow,

$$ D_n = \frac{Q_{An}}{Q_{Bn}} \tag{21} $$

As the results of numerical calculation, the following character is indicated.

1. The higher the story became, the smaller lateral force distribution ratio became,

2. The distribution ratio took a negative value at the higher story of the structure which had 9 or 10 stories.

3. The fewer the story order became, the larger the distribution ratio at 1st story became.

4. At the story of the same order from the top, the fewer the story order became, the larger the distribution ratio became.

5. At the lower story, the shearing deformation was large, and at the higher story, the bending deformation was large. (see, Fig. 4,5,6,7).

(2.2) Stiffness and Lateral Force Distribution on Continuous Arrangement of Aseismatic Walls.

Same as in the bracings, we may obtain the equation to give R_n on this case. In A frame, instead of A_n that is the column area, the equivalent column area A'_n is adopted and is given approximately as follow,

$$A'_n = A_{An} + \frac{A_{wn}}{6} \tag{22}$$

where, A_{wn} = horizontal sectional area of the aseismatic wall ($t_a \cdot \ell$), consequently, the bending softness coefficient ${}_mK_A$ is,

$${}_mK_A = \frac{2h^2}{AE\ell^2} \tag{23}$$

also, the shearing softness coefficient ${}_sK_A$ is,

$${}_sK_A = \frac{2k(1-\nu)}{E\,A_{wn}} \tag{24}$$

where, ν = Poisson's ratio

k = constant to be determined by the shape of the section.

Stiffness and lateral force distribution were the same results that had been obtained in the bracing. (see Fig. 8)

3. Stiffness and Lateral Force Distribution on Dispersive Arrangement of Aseismatic Elements.

(3.1) Calculation of Lateral Force Distribution Ratio by means of Simultanous Equation.

Consider the general aseismatic bracing's arrangement as shown in Fig. 9. The shearing Force acting at the n-th story in A frame is denoted by Q_{An}, and at the n-th story in B frame is denoted by Q_{Bn}, where,

$$Q_{An} + Q_{Bn} = Q_n \tag{25}$$

The relation between the angle of deflection caused by the bending deformation and the shearing force at the n-th story in A frame is

$${}_mR_{An} - {}_mR_{A(n-1)} = {}_mK_{An} \sum_n^{\infty} Q_{Ai} \tag{26}$$

where, ${}_mK_{An}$ = the bending softness at the n-th story in A frame

${}_mR_{An}$ = the angle of deflection caused by bending deformation at the n-th story in A-frame

Similarly, at the n-th story in B frame

$$ {}_mR_{Bn} - {}_mR_{B(n-1)} = {}_mK_{Bn} \sum_{n}^{s} Q_{Bi} \tag{27} $$

where, ${}_mK_{Bn}$ = the bending softness at the n-th story in B frame

${}_mR_{Bn}$ = the angle of deflection caused by bending deformation at the n-th story in B frame.

Also, the relation between the angle of deflection caused by the shearing deformation and the shearing force at the n-th story in A frame is given by the following equation,

$$ {}_sR_{An} = {}_sK_{An} \cdot Q_{An} \tag{28} $$

where, ${}_sK_{An}$ = the shearing softness at the n-th story in A frame

${}_sR_{An}$ = the angle of deflection caused by the shearing deformation at the n-th story in A frame

Similarly, at the n-th story in B frame.

$$ {}_sR_{Bn} = {}_sK_{Bn} \cdot Q_{Bn} \tag{29} $$

The total angle of deflection is obtained by summing the angle of deflection caused by the bending deformation and the shearing deformation, and that is

$$ {}_mR_{An} + {}_sR_{An} = R_{An} \tag{30} $$

and,
$$ {}_mR_{Bn} + {}_sR_{Bn} = R_{Bn} \tag{31} $$

then, as, in Eq.(30) and (31), $R_{An} + R_{Bn}$

$$ {}_mR_{An} + {}_sR_{An} = {}_mR_{Bn} + {}_sR_{Bn} \tag{32} $$

${}_mR_{A1}, {}_mR_{A2}, \cdots\cdots\cdots, {}_mR_{An}, \cdots, {}_mR_{AS}$ can be required from Eq.(26). Similarly, ${}_mR_{B1}, {}_mR_{B2}, \cdots\cdots\cdots, {}_mR_{Bn}, \cdots, {}_mR_{Bs}$ are required from Eq. (27). Substituting the value of them, Eq.(28) and Eq.(29) in Eq.(32), the following equation at the n-th story may be obtained,

$$ \begin{aligned} &\left(\sum_{1}^{n} {}_mK_{Ai} + \sum_{1}^{n} {}_mK_{Bi}\right) Q_{AS} + \left(\sum_{1}^{n} {}_mK_{Ai} + \sum_{1}^{n} {}_mK_{Bi}\right) Q_{A(S-1)} + \cdots\cdots\cdots \\ &+ \left(\sum_{1}^{n} {}_mK_{Ai} + \sum_{1}^{n} {}_mK_{Bi}\right) Q_{A(n+1)} + \left\{\left(\sum_{1}^{n} {}_mK_{Ai} + \sum_{1}^{n} {}_mK_{Bi}\right) + \left({}_sK_{An} + {}_sK_{Bn}\right)\right\} Q_{An} \\ &+ \left(\sum_{1}^{n-1} {}_mK_{Ai} + \sum_{1}^{n-1} {}_mK_{Bi}\right) Q_{A(n-1)} + \cdots\cdots\cdots\cdots + \left({}_mK_{A1} + {}_mK_{B1}\right) Q_{A1} \\ &= \left(\sum_{1}^{n} {}_mK_{Bi}\right) Q_{S} + \left(\sum_{1}^{n} {}_mK_{Bi}\right) Q_{(S-1)} + \cdots\cdots\cdots + \left(\sum_{1}^{n} {}_mK_{Bi}\right) Q_{(n+1)} \\ &+ \left(\sum_{1}^{n} {}_mK_{Bi} + {}_sK_{Bn}\right) Q_{n} + \left(\sum_{1}^{n-1} {}_mK_{Bi}\right) Q_{(n-1)} + \cdots\cdots\cdots + \left(\sum_{1}^{2} {}_mK_{Bi}\right) Q_{2} \\ &+ \left({}_mK_{B1}\right) Q_{1} \end{aligned} \tag{33} $$

Consequently, the shearing force at each story in A and B frame may be required from setting up the simultaneous equations to be equal to the story orders in a number, and then lateral force distribution ratio may be determined from Eq.(21).

As the results of numerical calculation, the following facts is indicated.

1. In the continuous arrangement of aseismatic elements,

the lateral force distribution ratio was smaller than what considered only the shearing deformation at the same story on the higher portion. In the structure that had many stories, the distribution ratio at the higher story took a negative value.

2. When aseismatic elements was arranged dispersively at one story at least, the distribution ratio at each story was larger than that at the same story on the above condition 1. But, the parts of continuous arrangement kept the the character of itself.

3. In uniformly dispersive arrangements of aseismatic elements, the distribution ratio at each story was nearly equal to what considered only the shearing deformation. (see Fig. 9,10,11,12,13)

(3.2) Comparison of Stiffness on Arrangements with Steel Model.

The stiffness may be compared on arrangements of aseismatic bracings with the model of a steel framed structure in which is inserted bracings, as shown in Fig. 14,15. The equivalent external force is given parallel to the ridge direction at each story, so that the following facts is indicated.

1. In the continuous arrangement of aseismatic bracings, the deformation is the largest of all, that is to say, the stiffness is the smallest.

2. In the uniformly dispersive arrangement of aseismatic bracings, the deformation is the smallest, that is to say, the stiffness is the largest.

3. In the arrangements which exist between 1 and 2, the deformation exists between 1 and 2, so that the stiffness exists between 1 and 2. (see Fig. 16)

4. Stiffness and Lateral Force Distribution on Arbitrary Arrangement of Aseismatic Elements by means of Stress-Controlled Calculation.

In the case that aseismatic elements are arranged dispersively, the determination of lateral force distribution is required from setting up the simultaneous equations about the each of arrangements as discussed in the former chapter. But, that method is difficult, and then the practical method will be introduced in this chapter. This is the method to require lateral force distribution from being aware of both the stress of aseismatic elements and columns on arbitrary arrangements of aseismatic elements. This method is styled "Stress-Controlled Calculation".

Stress-Controlled Calculation may express the bending deformations on arrangements of aseismatic elements with the axial stress of the each of columns, and their shearing deformations are expressed with the axial stress of aseismatic elements. Then, observing the each stress discribed in the above, the calculation may be put gradually foreward. In this method, basing the aseismatic elements and the shear-

ing stiffness of general frames, lateral force distribution of arbitrary arrangements may be required by multiplying the correct coefficients caused by the relative bending deformation.

Consequently, to determine lateral force distribution is comparatively easy in any arrangement. Also, through the medium of the stress of the structural elements, the unreasonable stress condition may be discovered on a way of the calculation, so that, reasonable lateral force distribution may be determined by controlling the stress with the alteration of the cross section. According to this method, to determine a cross section is be possible, too, if the distribution has been known.

(4.1) Stiffness and Lateral Force Distribution by means of Stress-Controlled Calculation on Arbitrary Arrangement of Bracings in Elastic or Plastic Range.

In this chapter, we will derive the fundamental expression to determine lateral force distribution on aseismatic bracings which are symmetric arrangement in elastic or plastic range. Now, when aseismatic bracings to be arranged at the n-th story in A frame have reached the yield stress σ_o, and exist in the perfectly plastic range, the angle of deflection based on the shearing deformation is, (Fig. 17)

$$ {}_sR_{An} = K_{on} \cdot {}_sR_o = K_{on} \cdot \frac{2\sigma_o}{E \sin 2\theta_n} \tag{34} $$

$$ \text{where, } K_{on} = \frac{{}_sR_{An}}{{}_sR_o} \tag{35} $$

Then, the total angle of deflection at the n-th story in A or B frame, that is R_{An} or R_{Bn}, may be given as below,

$$ R_{An} = \left(\frac{2\sigma_o}{E \sin 2\theta_n}\right) K_{on} + \frac{2}{E\ell} \sum_{i=1}^{n} \sigma_{Ai} \cdot h_i \tag{36} $$

$$ R_{Bn} = \frac{h_n^2}{2EA_{Bn}^2} Q_{Bn} + \frac{2}{E\ell} \sum_{i=1}^{n} \sigma_{Bi} \cdot h_i \tag{37} $$

where, $R_{An} = R_{Bn}$ and if $A_{An} = A_{Bn} = A_n$,

$$ \left\{\left(\frac{2\sigma_o}{\sin 2\theta_n}\right) K_{on} + \frac{2}{\ell} \sum_{i=1}^{n} (\sigma_{Ai} - \sigma_{Bi}) h_i\right\} A_n^2 = \frac{h_n^2}{2} Q_{Bn} \tag{38} $$

Then, Eq.(38) gives the following equation for lateral force distribution at the n-th story D_n,

$$ D_n = \frac{Q_{An}}{Q_{Bn}} = \frac{{}_sK_{Bn}}{{}_sK_{An}} \cdot \mu_{(AB)n} = \frac{{}_sD_{An}}{{}_sD_{Bn}} \mu_{(AB)n} \tag{39} $$

where, ${}_sK_{An}$ = shearing softness coefficient at the n-th story in A frame $\left(\frac{1}{{}_sD_{An}}\right)$

${}_sK_{Bn}$ = shearing softness coefficient at the n-th story in B frame $\left(\frac{1}{{}_sD_{Bn}}\right)$

${}_sD_{An}$ = lateral force distribution coefficient of shearing deformation at the n-th story in A frame

${}_sD_{Bn}$ = lateral force distribution coefficient of shearing deformation at the n-th story in B frame.

$\mu_{(AB)n}$ = the correct coefficient caused by the relative bending deformation of lateral force distribution of B frame for that of A frame at the n-th story.

where, $\mu_{(AB)n}$ is given as below,

$$\mu_{(AB)n}=\frac{1}{\left[K_{on}+\frac{\sin 2\theta_n}{l}\cdot\frac{\sum_{i=1}^{n}(\sigma_{Ai}-\sigma_{Bi})h_i}{\sigma_o}\right]} \tag{40}$$

When the axial stress of the aseismatic bracings at the n-th story is denoted by σ_{dn}, the following expressions is obtained,

$$\left.\begin{aligned}&\text{if } \sigma_{dn}\leqq\sigma_o \text{ (in elastic range)} \cdots\cdots\cdots \sigma_o=\sigma_{dn},\ K_{on}=1\\&\text{if } \sigma_{dn}>\sigma_o \text{ (in plastic range)} \cdots\cdots\cdots \sigma_{dn}=\sigma_o,\ K_o=\frac{{}_sR_{An}}{{}_sR_o}>1\end{aligned}\right\} \tag{41}$$

in the above, if $h_i=h$, K_{on} is

$$K_{on}=\frac{{}_sR_{An}}{{}_sR_o}=\frac{2\sin^2\theta}{\sigma_o}\left[\frac{lh}{2A_n}\left(\frac{Q_{Bn}}{2A_n}\right)-\sum_{i=1}^{n}(\sigma_{Ai}-\sigma_{Bi})\right] \tag{42}$$

With the above way, lateral force distribution of bracings in elastic or plastic range may be required. That is to say, when $\mu_{(AB)n}=1$ is assumed, the 1st approximate value of D_n may be given from Eq.(40). The distribution is obtained by repeating the similar calculation. Next, we derive the fundamental expression to determine the distribution on aseismatic elements which are asymmetric arrangement.

In Fig. 18, the center of rotation is a point O. The distance between that point and the each of A,B,C,D,1,2, at the n-th story is denoted by X_{An}, X_{Bn}, X_{Cn}, X_{Dn}, Y_{1n}, Y_{2n}. The relation between the total angle of deflection R_{An} and R_{Bn} at the n-th story in A,B frame is shown below,

$$R_{An}=R_{Bn}\left(\frac{X_{An}}{X_{Bn}}\right)=R_{Bn}\cdot X_{(AB)n} \tag{43}$$

then, when the bracings which arrange at the n-th story in A,B frame is in plastic range, R_{An} and R_{Bn} are shown generally below,

$$R_{An}={}_sK_{An}\cdot Q_{An}\cdot K_{on}-\frac{2}{El}\sum_{i=1}^{n}\sigma_{Ai}h_i \tag{44}$$

$$R_{Bn}={}_sK_{Bn}\cdot Q_{Bn}+\frac{2}{El}\sum_{i=1}^{n}\sigma_{Bi}h_i \tag{45}$$

Substituting Eq.(44) and Eq.(45) in Eq. (43), lateral force distribution $D_{(AB)n}$ is given below,

$$\begin{aligned}D_{(AB)n}&=\frac{Q_{An}}{Q_{Bn}}=\frac{{}_sD_{An}}{{}_sD_{Bn}}\mu_{(AB)n}=\frac{{}_sD_{An}}{{}_sD_{Bn}}\cdot X_{(AB)n}\mu'_{(AB)n}\\&=D'_{(AB)n}X_{(AB)n}\end{aligned} \tag{46}$$

where,

$$D'_{(AB)n}=\frac{{}_sD_{An}}{{}_sD_{Bn}}\mu'_{(AB)n}$$

$$\mu'_{(AB)n}=\frac{1}{\left[K_{on}+\frac{2\,{}_sD_{An}}{El}\left\{\frac{\sum_{i=1}^{n}\sigma_{Ai}h_i-\left(\sum_{i=1}^{n}\sigma_{Bi}h_i\right)X_{(AB)n}}{Q_{An}}\right\}\right]} \tag{47}$$

then, K_{on} and $X_{(AB)n}$ are given as below.

$$K_{on}=\left\{\frac{{}_sD_{An}}{{}_sD_{Bn}}X_{(AB)n}\cdot\left(\frac{Q_{Bn}}{Q_{An}}\right)-\frac{2\,{}_sD_{An}}{E\ell}\left\{\frac{\sum_{i=1}^{n}\sigma_{Ai}h_i-\left(\sum_{i=1}^{n}\sigma_{Bi}h_i\right)X_{(AB)n}}{Q_{An}}\right\}\right| \tag{48}$$

$$X_{Bn}=\left\{\frac{3D'_{(AB)n}+D'_{(CB)n}+6D'_{(DB)n}+2D'_{(IB)n}\left\{\frac{y_{in}(y_{in}-y_{2n})}{\ell_x^2}\right\}}{1+3D'_{(AB)n}-D'_{(CB)n}-3D'_{(DB)n}}\right\}\ell_x \tag{49}$$

Like the above, the expressions to determine lateral force distribution in the case of accompany with a rotation are somewhat complicated. But, the distribution may be required from the repeated calculation to have been discussed in former. The numerical calculation is shown in Fig. 19. This result corresponds to what has been discussed in the former chapter.

(4.2) Stiffness and Lateral Force Distribution by means of Stress-Controlled Calculation on Arbitrary Arrangement of Aseismatic Walls in Elastic or Plastic Range.

The fundamental equation to determine lateral force distribution on aseismatic walls of the symmetric arrangement in elastic or plastic range, may be required similarly with the way of bracings. (Fig. 20) Now, when the aseismatic walls to be arranged at the n-th story in A frame have reached the shear yield stress and exist at the perfectly plastic range, the angle of deflection based on the shearing deformation is,

$${}_sR_{An}=K_{on}\cdot{}_sR_o=K_{on}\cdot 2(1+\nu)\frac{\tau_o}{E} \tag{50}$$

where, $k=1$ is assumed. In this case, the total angle of deflection at the n-th story in A, B frame, that is R_{An}, R_{Bn}, is given as below,

$$R_{An}=\left\{\frac{2(1+\nu)\tau_o}{E}\right\}K_{on}+\frac{2}{E\ell}\sum_{i=1}^{n}\sigma_{Ai}h_i \tag{51}$$

$$R_{Bn}=\frac{k_n^2}{2EA_{Bn}^2}Q_{Bn}+\frac{2}{E\ell}\sum_{i=1}^{n}\sigma_{Bi}\cdot h_i \tag{52}$$

where, $R_{An}=R_{Bn}$ and if $A_{An}=A_{Bn}=A_n$

$$\left\{2(1+\nu)\tau_o K_{on}+\frac{2}{\ell}\sum_{i=1}^{n}(\sigma_{Ai}-\sigma_{Bi})h_i\right\}A_n^2=\frac{k_n^2}{2}Q_{Bn} \tag{53}$$

Then, Eq.(53) gives the following equation to determine the distribution at the n-th story,

$$D_n=\frac{Q_{An}}{Q_{Bn}}=\frac{{}_sK_{Bn}}{{}_sK_{An}}\mu_{(AB)n}=\frac{{}_sD_{An}}{{}_sD_{Bn}}\mu_{(AB)n} \tag{54}$$

where, the correct coefficient is

$$\mu_{(AB)n}=\frac{1}{\left[K_{on}+\frac{1}{(1+\nu)\ell}\cdot\frac{\sum_{i=1}^{n}(\sigma_{Ai}-\sigma_{Bi})h_i}{\tau_o}\right]} \tag{55}$$

When the axial stress acting on the aseismatic walls at the n-th story is denoted by τ_{wn}.

$$\text{if } \tau_{wn}\leqq\tau_o\text{(in elastic range)}\text{-----}\tau_o=\tau_{wn},\ K_{on}=1\ \Big\} \tag{56}$$

if $\tau_{wn} > \tau_o$ (in plastic range) $\cdots\cdots \tau_{wn} = \tau_o, K_o = \frac{sR_{An}}{sR_o} > 1$)

Also, if $k_i = k$, K_{on} is given as below,

$$K_{on} = \frac{sR_{An}}{sR_o} = \frac{k}{(1+\nu)l} \frac{1}{\tau_o} \left[\frac{lk}{2A_n} \left(\frac{Q_{Bn}}{2A_n} \right) - \sum_{i=1}^{n} (\sigma_{Ai} - \sigma_{Bi}) \right] \quad (57)$$

Like the above, lateral force distribution on aseismatic walls in elastic or plastic range, may be required. The method of this calculation corresponds to that of bracings. The fundamental equation to determine lateral force distribution on asymmetric arrangement may be treated as Eqs. (46)~(49).

The numerical calculation is shown in Fig. 21.

(4.3) Comparison of Experimental Results of Achlilite Model and Results of Stress-Controlled Calculation.

We indicate the experimental results on stiffness and lateral force distribution with the achlilite to hold bracings, as shown in Fig. 22. When the horizontal load apply parallel to the plane direction of the arrangement shown in Fig. 23, stiffness of each type which is expressed by the horizontal displacement at the highest story, is shown in Fig. 24. The ordinate shows the number of bracings to be arranged in the each of ridge direction that has 3 spans. According to this results, the maximum horizontal displacement was shown at the continuous arrangement, and the minimum value at the uniformly dispersive arrangement. Fig. 25 (a) ~ (d) show lateral force distribution modulus λ_n on the arrangements of beam direction. In that figure, solid lines show experimental values and dotted lines are values by Stress-Controlled Calculation. According to the above, the profit of dispersive arrangements has been indicated, and the experimental results almost correspond to the values of calculation. The above does not contradict to the results with the steel model or the numerical calculation discussed in the former chapter.

(4.4) Fundamental Equations of Stress-Controlled Calculation on Reinforced Concrete Construction.

Considering the character of reinforced concrete which contains the shearing deformation of aseismatic elements and the bending deformation mechanism on the whole structure, in order to revise the equivalent axial stress of bracings and columns, we introduce newly the correct coefficient of shearing stiffness and bending stiffness. And then, Stress-Controlled Calculation is revised like the form which is possible to express the change of lateral force distribution caused by arrangements with the change of the external force.

The above result is shown as follows,

$$D_n = \frac{Q_{An}}{Q_{Bn}} = \frac{(sD_{An})_1}{(sD_{Bn})_1} \mu_{(AB)n} \quad (58)$$

$$\mu_{(AB)n}=\frac{\frac{{}_sk_{An}}{{}_sk_{Bn}}\cdot X_{(AB)n}}{\left\{1+\left(\frac{\sin 2\theta_n}{2l}\right)\frac{\sum_{i=1}^{n}\left(\frac{\sigma_{A2i}}{{}_mk_{A2i}}-\frac{\sigma_{A1i}}{{}_mk_{A1i}}\right)h_i-\left\{\sum_{i=1}^{n}\left(\frac{\sigma_{B2i}}{{}_mk_{B2i}}-\frac{\sigma_{B1i}}{{}_mk_{B1i}}\right)h_i\right\}X_{(AB)n}}{\frac{\sigma_{dn}}{{}_sk_{An}}}\right\}} \quad (59)$$

where, ${}_sk_{An}$=correct coefficient of shearing stiffness at the n-th story in A-frme.

${}_sk_{Bn}$=correct coefficient of shearing stiffness at the n-th story in B frame.

${}_mk_{A2i}$=correct coefficient of bending stiffness on compressive axial force of column at the i-th story in A frame.

${}_mk_{B2i}$=correct coefficient of bending stiffness on compressive axial force of column at the i-th story in B frame.

${}_mk_{A1i}$=correct coefficient of bending stiffness on tensile axial force of column at the i-th story in A frame.

${}_mk_{B1i}$=correct coefficient of bending stiffness on tensile axial force of column at the i-th story in B frame.

σ_{A2i}=compressive axial stress of column at the i-th story in A frame.

σ_{B2i}=compressive axial stress of column at the i-th story in B frame.

σ_{A1i}=tensile axial stress of column at the i-th story in A frame.

σ_{B1i}=tensile axial stress of column at the i-th story in B frame.

$({}_sD_{An})_x$=initial shearing stiffness at the n-th story in A frame.

$({}_sD_{Bn})_x$=initial shearing stiffness at the n-th story in B frame.

The results of calculation on the continuous and dispersive arrangements is shown in Fig. 26. We may know the character of lateral force distribution on each arrangement with the change of external force.

5. Conclusion.

In this paper, at first, the continuous arrangement to have been adopted customarily was discussed and that lateral force distribution was required newly by finite difference and the contrary effect was indicated. At next, uniformly dispersive arrangements are especially effective, was indicated. At the later, Stress-Controlled Calculation was introduced, that is what may require easily lateral force distribution on arrangements of aseismatic elements. Through the examples, lateral force distribution on the dispersive arrangements was shown.

We think that the freedom on sturctural planning increase by means of using stiffness and lateral force distribution on the arrangements of aseismatic elements.

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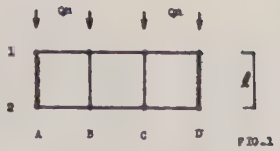


FIG.1

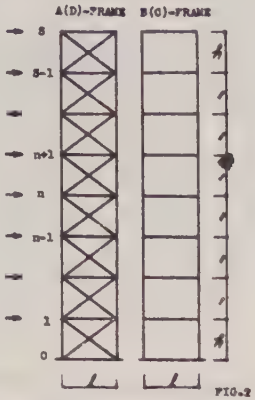


FIG.2

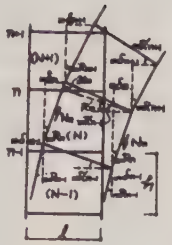


FIG.3

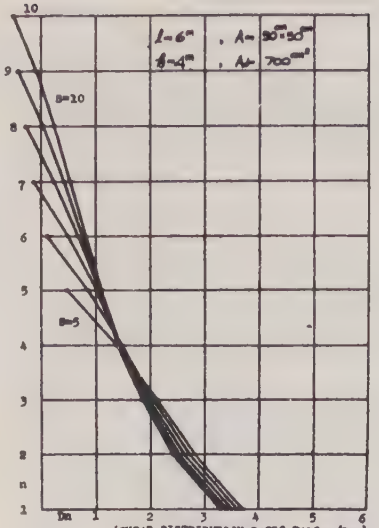


FIG.4 TENDENCY OF LATERAL FORCE DISTRIBUTION IN FRAME

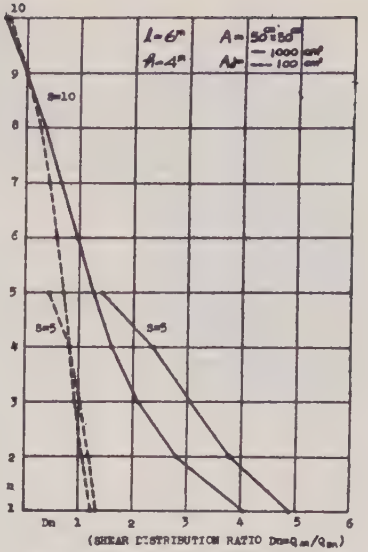


FIG.5.TENDENCY OF LATERAL FORCE DISTRIBUTION IN FRAME

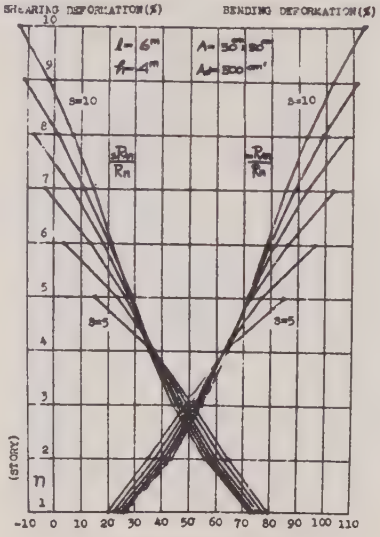


FIG.6.RELATION BETWEEN SHEARING AND BENDING DEFORMATION

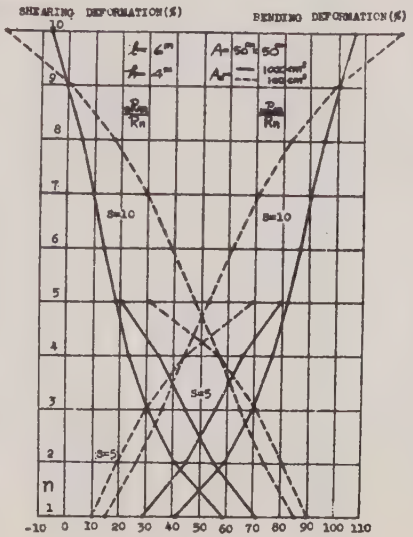


FIG.7.RELATION BETWEEN SHEARING AND BENDING DEFORMATION

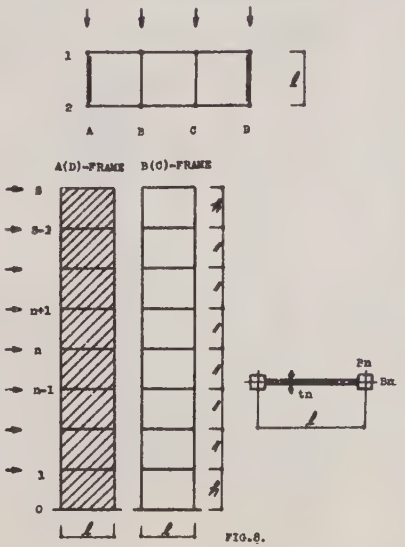


FIG.8.

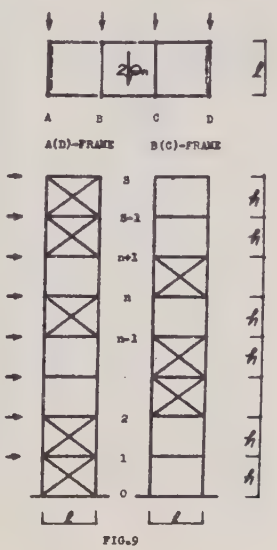


FIG.9

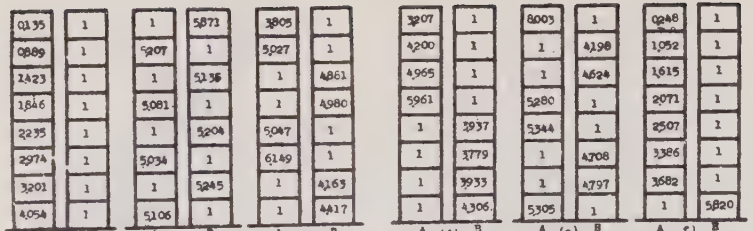


FIG.10. LATERAL FORCE DISTRIBUTION RATIO ON BRACINGS OF UNIFORM SECTION ARRANGED DISPERSIVELY

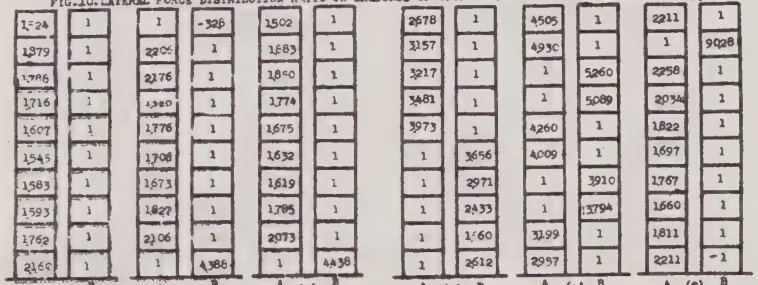


FIG.11. LATERAL FORCE DISTRIBUTION RATIO ON BRACINGS OF VARIABLE SECTION ARRANGED DISPERSIVELY

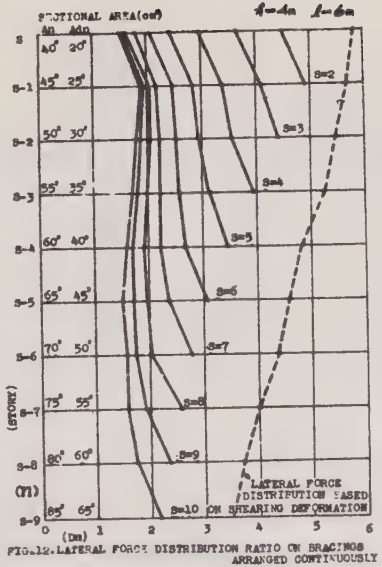


FIG.12. LATERAL FORCE DISTRIBUTION RATIO ON BRACINGS ARRANGED CONTINUOUSLY

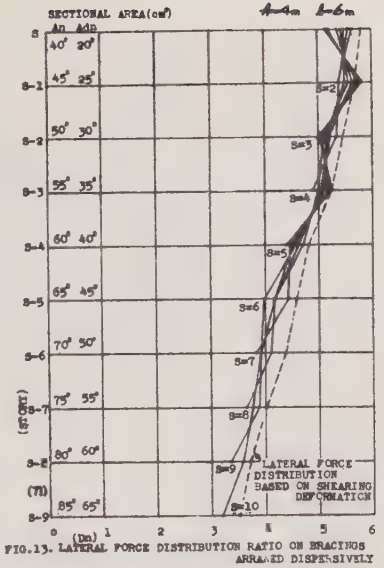
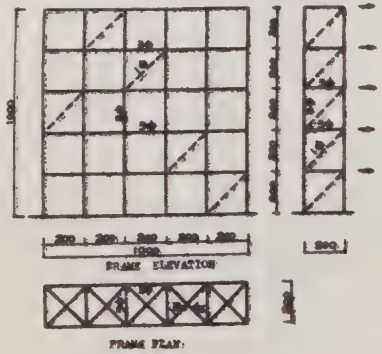


FIG.13. LATERAL FORCE DISTRIBUTION RATIO ON BRACINGS ARRANGED DISPERSIVELY



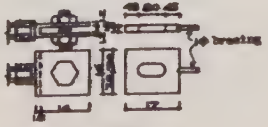


FIG.14. STEEL MODEL

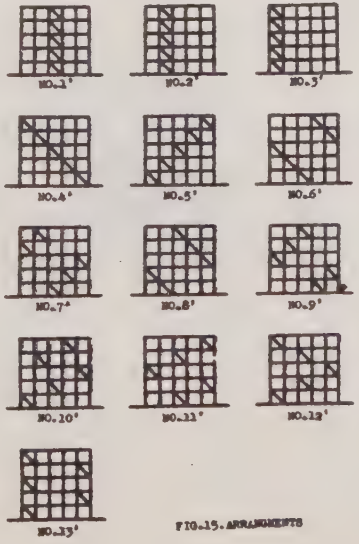


FIG.15. ARRANGEMENTS

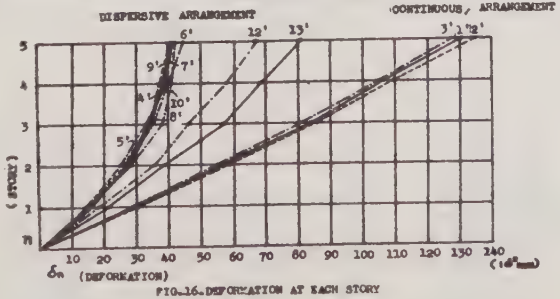


FIG.16. DEFORMATION AT EACH STORY

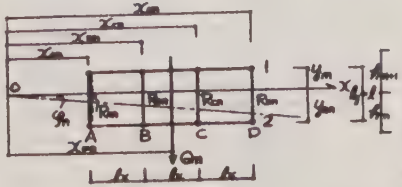


FIG.18. DISPLACEMENT OF ROTATION

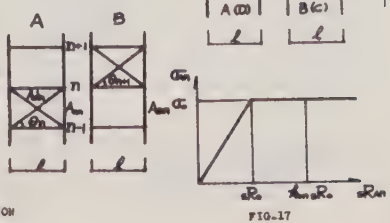


FIG.17

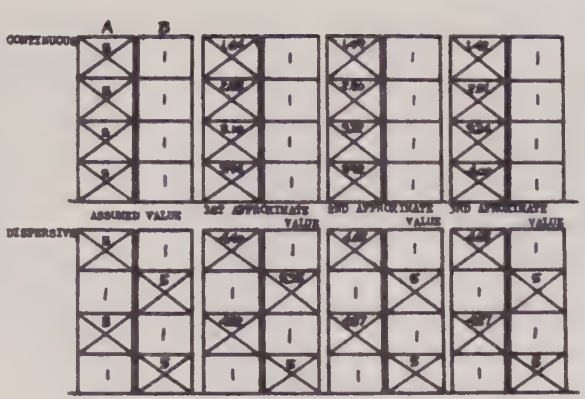


FIG.19.RESULTS OF STRESS-CONTROLLED CALCULATION ON SPACINGS

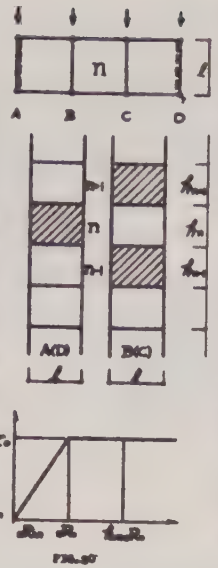


FIG.20

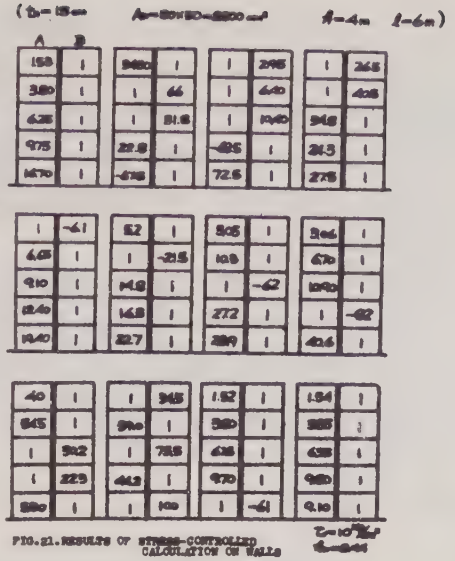


FIG.21.RESULTS OF STRESS-CONTROLLED CALCULATION ON WALLS

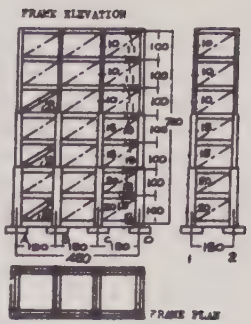


FIG.22 ACHILITY MODEL (UNIT mm)

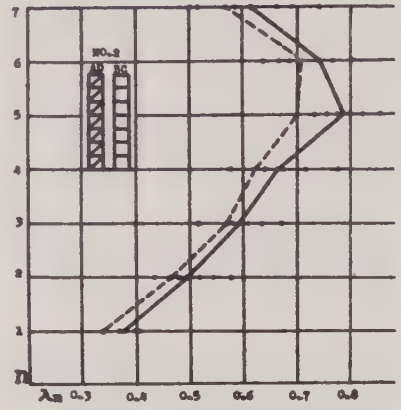


FIG.23.LATERAL FORCE DISTRIBUTION MODULUS (a)

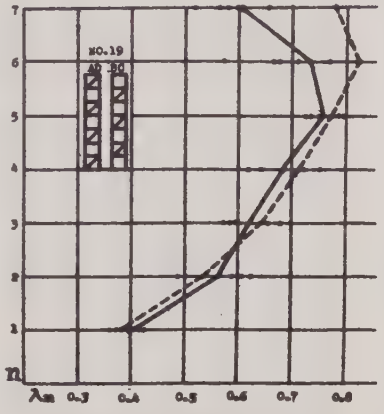


FIG.23.LATERAL FORCE DISTRIBUTION MODULUS (b)

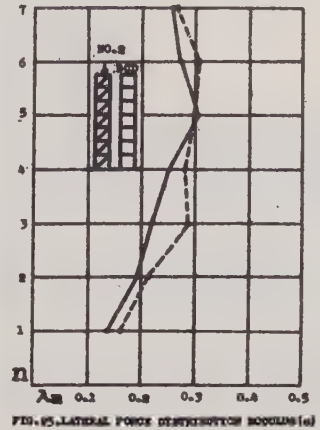


FIG.25.LATERAL FORCE DISTRIBUTION MODULUS(c)

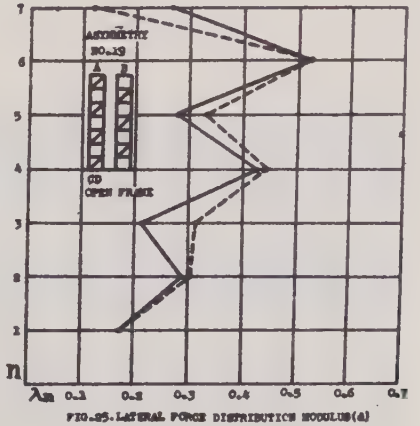


FIG.25.LATERAL FORCE DISTRIBUTION MODULUS(d)

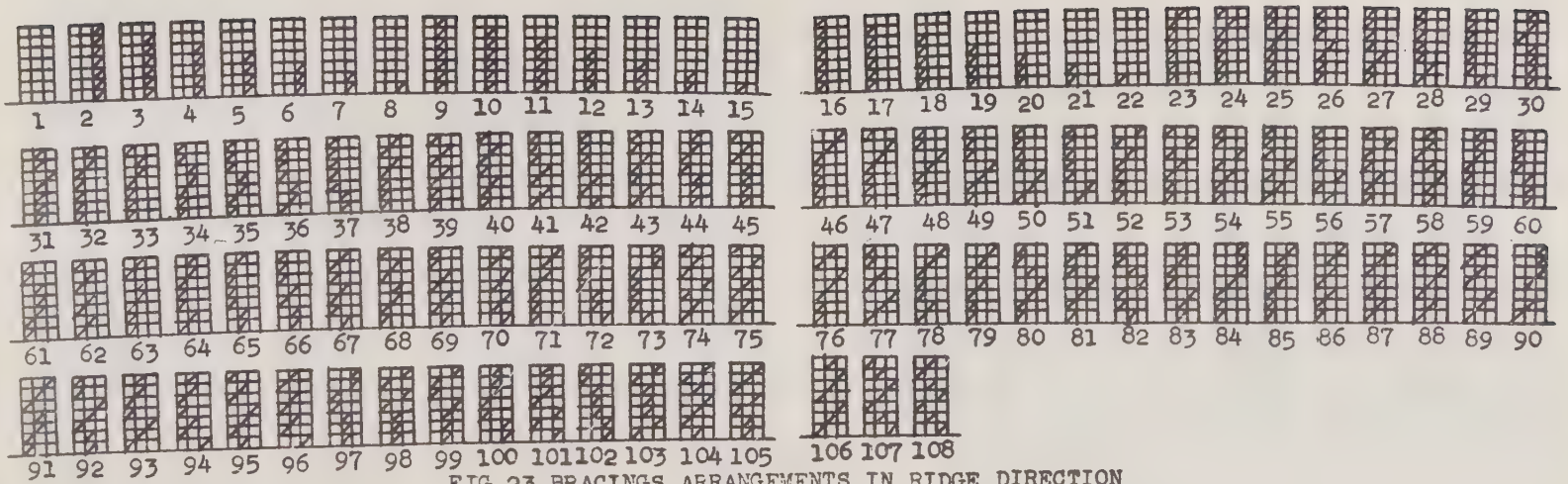


FIG.23.BRACINGS ARRANGEMENTS IN RIDGE DIRECTION

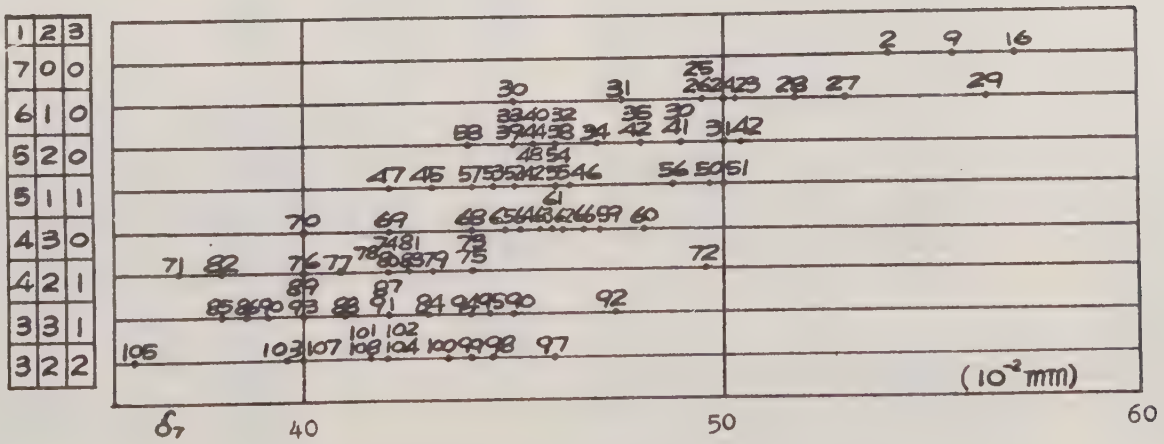


FIG.24.HORIZONTAL DISPLACEMENT AT 7-th STORY

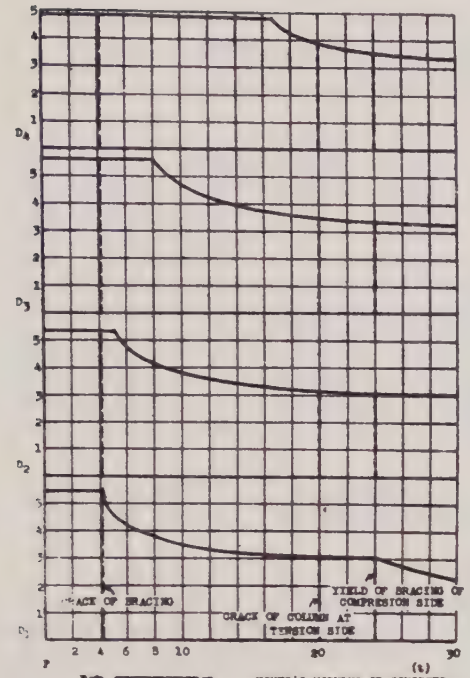


FIG.26.-LATERAL FORCE DISTRIBUTION RATIO D_4 ON DISPERSIVE ARRANGEMENT (b)
(REINFORCED CONCRETE STRUCTURE)

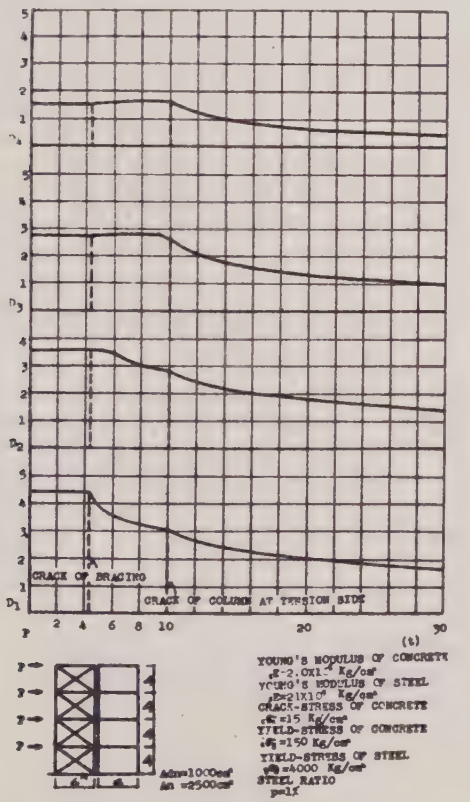


FIG.26.-LATERAL FORCE DISTRIBUTION RATIO D_4 ON CONTINUOUS ARRANGEMENTS (a)
(REINFORCED CONCRETE STRUCTURE)

BEHAVIOR AND DESIGN OF WOODEN BUILDINGS SUBJECTED TO EARTHQUAKE

By

Kenzaburo TAKEYAMA*, Toshihiko HISADA* and Yorihiko OHSAKI*

1 INTRODUCTION

In Japan wooden construction is a traditional one dating back 2000 years and about 90 percent of present Japanese buildings are constructed of wood. Therefore, almost all damage to buildings caused by the severe earthquakes in the past was damage to buildings of this type of construction and there have been made extensive investigation in this connection [1] from which the features of damage to the buildings have been clarified.

Construction of Japanese Wooden Buildings. The typical construction of Japanese wooden buildings is described below.

1) The column and horizontal member composing the frame are connected with each other by skilfully made tenon joints as shown in Figs. 1 and 2, and no metal connectors such as bolts or steel plates are, in general, employed in the construction. Diagonal bracings are not so extensively used, although their use as a means against earthquake forces has gradually increased during these thirty years.

2) The wall panel is usually the so-called "Shinkabe" which is made by covering bamboo lath peculiar to Japan with mud plaster and is thinner than the pillars around it; sometimes the wall is finished with Japanese style wooden sheathings. In recent years, foreign style wooden sheathings finished with mortar over metal lath have come to be used. This type of wall is called "Okabe" in Japanese.

3) The roof materials of buildings are Japanese tiles, cement tiles, asbestos cemente slates and metal plates, and thatched roofs are sometimes seen in the rural area. The weight of a heavy roof is about twice that of a light roof.

Characteristics of Earthquake Damage. The typical features of earthquake damage to wooden buildings are as follows:

1) At the joint connecting the column and girder, the failure usually occurs as shown in Figs. 1** and 2, accompanying the falling of finishings as shown in Fig. 3. As the inclination of the structure increases, its restoring force against distortion decreases due to the structural deterioration and roof weight and finally becomes negative, which results in the complete collapse of the building as shown in Fig. 4.

---

* Building Research Institute, Ministry of Construction, Japanese Government

**Figs. 1, 2, 3 and 4 were kindly offered by courtesy of Architectural Department, Faculty of Eng., and Earthquake Research Inst., Univ. of Tokyo.

2) In case of two story buildings, the first story structure usually suffers more damage and it is often seen that the lower structure falls down on the ground while the upper one survives the earthquake shock.

3) Earthquake damage to wooden buildings is considerably influenced by the ground on which they stand, and, in general, the softer is the subsoil, the greater is the rate of damage to the buildings. Furthermore, the damage due to differential settlements of foundations is observed for the wooden buildings standing on soft ground.

4) Sliding of the building as a whole is sometimes seen when there are no anchor bolts connecting the sill to the foundation. Also, the damage to superstructure is often observed when the foundation construction is poor to resist seismic force.

5) The relation between earthquake damage to wooden buildings and the maximum acceleration of ground motion obtained from several past large earthquakes is as follows:[2]

0.1g: cracking and falling-off of the wall finishings.

0.2g: slight distortion of structural frame with some deterioration of the connecting joints.

0.3g: most houses inclined, a few collapsed.

0.4g: most houses destroyed.

Vibrational Characteristics of Wooden Buildings. Dynamical behaviors of wooden buildings obtained from past investigations are summarized below.

1) Natural periods in small amplitude vibrations are generally in the range of 0.18 to 0.4 sec for one story buildings and of 0.25 to 0.6 sec for two story buildings. These variations are due to the roof weight and structural rigidity which mainly depends on the quantity of frames with walls or bracings.

2) The natural periods mentioned above increase with the amplitudes of vibration, amounting to twice to three times the said periods in the case of large amplitude vibrations.

3) During earthquakes the buildings vibrate with their fundamental periods and their amplitudes become large when those periods are close to the predominant periods of the ground on which they stand.

4) Fractions of critical damping observed in small amplitude vibrations of wooden buildings are in the range of 0.05 to 0.15.

2 GROUND CONDITION AND EARTHQUAKE DAMAGE

It is a well known fact that the damage of wooden buildings due to earthquake is closely related with ground conditions and the damage rate is evidently high where the alluvial soil is deeply deposited[3]. This fact is supposedly attributable to two reasons: (1) because of the characteristics of a soft ground that the periods of vibration extend over quite

a wide range without a distinct predominant period and the ground motion lasts for a long time, a building continues to be in a state of quasi-resonance with the ground even after it has been partially damaged and, consequently, its natural period has been lengthened, and (2) because of the insufficient bearing capacity of the ground, a large differential settlement takes place at the foundation of the building so that it induces excessive stresses in the superstructure which accelerate the collapse of the building.

In the following, the damage rates of wooden buildings in the Fukui Earthquake of 1948 and the Great Kanto Earthquake of 1923 will be reviewed from this standpoint and will be presented as an evidence of the interrelationship between the earthquake damage and the ground conditions.

Fukui Earthquake. With respect to each of 60 villages in and around the Fukui Plain, the damage rate of wooden buildings, i.e. the ratio of the number of collapsed buildings to the total number of existing buildings, was investigated and the ground conditions of all investigated villages were divided into four categories:

I. Entire village is located upon the Alluvium

II. Approximately one-half of the village is located upon the alluvium and the other half upon the Bed Rock

III. Bed Rock, geologically older than the Tertiary

VI. Sand Dune

The distance from the epicenter to each investigated village is in the range of 3.1 to 15.5 km and, with respect to the Fukui Earthquake, another investigation[4] revealed that the relation between the damage rate and the distance from the epicenter could statistically be given by the equation

$$y = 1800R^{-2.26}$$

Then, in accordance with this equation, the damage rate in each village has been modified as if it were at the distance R = 7.5 km from the epicenter in order to eliminate the effect of the distance. If the damage rates thus modified are plotted against the four categories of the ground condition, Fig. 5 is obtained.

The exact distribution of the depth of the Alluvium in the Fukui Plain and geotechnical properties thereof are not known. Nevertheless, Fig. 5 shows an evident tendency that, in general, the damage rate upon the Alluvium is far larger than that upon the Bed Rock, and that the damage rate in a village located on the Sand Dune is intermediate between the Alluvium and the Bed Rock.

Great Kanto Earthquake. Information on the number of wooden buildings in every block of Tokyo city collapsed and so heavily destroyed beyond any repair by the Great Kanto Earthquake is made available from the report of survey written by T. Matsuzawa[5], and by taking the ratio of these buildings to the total number of existing buildings at the date of the Earthquake the damage rate in each bolck has been determined.

On the other hand, for information on the thickness of Alluvial deposit under each block, the Subsoil Map of Tokyo[6] has been resorted to. The Alluvial deposit in Tokyo area consists of sandy, silty and clayey materials and is highly varied, and mechanical properties thereof are in the ranges shown in the following Table.

	Sandy	Silty	Clayey
Number of Sample	130	90	99
Cohesion (kg/cm^2)	0.06 - 0.26	0.12 - 0.34	0.09 - 0.35
Angle of Internal Friction (degree)	14 - 33	6 - 21	2 - 8

The total number of blocks, with which both the damage rate and the thickness of Alluvial deposit have thus been made known, is 1,464. Then, by sorting these data into intervals of 5 m of Alluvial thickness and by computing the averages of the damage rate and the Alluvial thickness within each interval, small circles in Fig. 6 can be obtained.

From Fig. 6 it will be seen immediately that the damage rate increases with the increase of the thickness of Alluvial deposit and, where the thickness exceeds about 30 m, the damage rate shows a tendency to increase rapidly. If we assume an equation of the type $y = ab^x + c$ for representing the relationship between the damage rate and the thickness of Alluvial deposit and determine unknown constants by the method of least square, the following equation can be obtained:

$$y = 0.982(1.086)^x + 0.26$$

Recently, H. Kawasumi has made an analysis to disclose the effects of individual, constituent layers of the Tokyo subsoil to the magnitude of seismic coefficient, and found that the effects of the layer of peat and the artificially filled top-soil are most remarkable.[7]

3 RELATION BETWEEN EARTHQUAKE DAMAGE TO WOODEN BUILDINGS AND QUANTITIES OF FRAMES THEREIN

In the great earthquake of Fukui of 1948, the authors investigated the relationship between the earthquake damage to wooden buildings and the quantities of framings with walls or bracings therein.[8] The total number of houses investigated was about 70, including one and two story buildings which were situated in and around Fukui City. They stand on deep Alluvium deposit and suffered severe shocks, especially in N-S direction, which caused various degrees of damage including slight and severe damage and complete destruction.

The results of investigation are shown in Figs. 7 and 8. These figures indicate the correlation between the features of damage to wooden buildings and the load carried by unit length of frame with wall therein, classified according to the kind of wall and its location in story.

In making those figures, the frames having diagonal bracings or walls with openings are calculated in terms of full wall panels; the rates follow the results obtained from static tests which are shown in Table 3. Although these figures represent considerable scattering, they lead to the following remarks:

1) The degree of damage increases as the load carried by unit frame length increases.

2) "Okabe" is expected to resist larger earthquake force than "Shinkabe".

3) The lateral force resistance of frame with wall located on the first floor of a two story building seems to be greater than that in one story building.

4) In the Fukui Plain, shocks in N-S direction might be stronger than those in E-W direction.

5) If the load carried by unit length of frame with "Shinkabe" in completely destroyed houses is assumed to be 300 kg/m or 540 kg/6 ft, the acceleration imposed on those houses might be in the range of 0.4 g to 0.6 g which can be estimated from Table 2 or Fig. 19.

4 STATIC AND DYNAMIC TESTS ON FRAMES WITH WALLS OR BRACINGS

It is considered that the principal lateral force resistance of wooden buildings comes from the frames with walls or bracings. Therefore several tests[9) 10) 11)] using models have been carried out to study the static and dynamic behavior of those resisting elements, but no experiment on full size frame with vertical loading has been made so far. The purpose of this test[12)] is to reveal the aseismic properties of the resisting frames which are commonly used in this country, using full-scale ones under real condition.

Frames with Walls or Bracings Tested. In Table 1 are listed all frames tested. Each pair of those was erected on a concrete floor and the weight corresponding to average vertical loading was imposed on the frames as shown in Figs. 9, 10, 11 and 12. The estimated vertical loads are 720 kg for 6 ft length frame, 1000 kg for 12 ft and 480 kg for 3 ft, respectively.

Static Tests. Horizontal racking load was alternatively applied to the beam connecting the tops of two frames erected in parallel up to the failure. Some of the results showing the relationship between horizontal load and distortion of the top of frame are illustrated in Figs. 13, 14, 15, 16, 17 and 18. From the test results, the relation between load and distortion for each frame is shown in Fig. 19. The correlation between dissipated work during one cycle loading and distortion of each frame is indicated in Fig. 20. The results of the static tests are summarized below.

1) The cracks of the wall finishings develop, in general, when the top displacements of the frames reach about 4 cm and then the finishings gradually fall off. A diagonal bracing with narrow section suffers buckling failure whereas one with wide section makes compressive failure of adjoining lumbers at both ends.

2) The strength of a frame with wall above the picture rail is about one-fourth that with full wall panel.

3) The strength of a frame without any bracing is very small and cannot be expected to resist earthquake force.

4) The strength and rigidity of "Okabe" are larger than those of "Shinkabe", and the superiority is based on the horizontal wood sheathing with nail joints.

5) The effectiveness of diagonal bracings is proved. Care should be taken to prevent their buckling failure as well as the compressive crushing of their adjoining lumbers.

6) The rigidity of a frame with wall panel is roughly proportional to the square of its length, i.e. the distance between two columns composing the frame.

7) As for the dissipated work during one cycle loading, the frames with diagonal bracings represent large value in comparison with other ones, and, in general, the energy to be absorbed by "Okabe" is larger than that by "Shinkabe".

Dynamic Tests. Free vibration tests of the frames were carried out at various loading stages during the static tests. Under alternating loads in the plus and in the minus directions, how the natural periods of the frames become longer with the increase of the amplitudes is shown in Fig. 21. Furthermore the relationship between equivalent fraction of critical damping and ratio of load to its maximum value is shown in Fig. 22. The results obtained from vibration tests are summarized below.

1) In general the natural periods of "Okabe" are less than those of "Shikabe".

2) The natural periods of frames with walls or bracings increase with the amplitudes of vibration. In case of "Okabe" the ratio of increase reaches twice to three times the periods for smaller amplitudes but it is not so remarkable in "Shinkabe". The natural periods of frames having bracings do not vary remarkably through the tests.

3) The equivalent fractions of critical damping of "Shinkabe" are in the range of 0.25 to 0.1, decreasing with increase of amplitudes due to the falling of infilled mud-plaster. On the other hand, the equivalent fractions of critical damping of "Okabe" increase with amplitudes due to the properties of nail joints for wood sheathings, ranging 0.1 to 0.3. Those values of frames with bracings are generally small and less than 0.1.

5 CONSIDERATIONS AND PROPOSAL OF ASEISMIC DESIGN

Permissible loads. The horizontal loads to be allowed for frames should be dertermined by considering the features of damage permissible which is considered as the falling of finishings after cracks develop. These loads, from the results of static tests, correspond to the displacements of 4 cm at the tops of frames, i.e. the angular deformation of 1/75 radian. This permissible load for each frame will be obtained from Fig. 19 as shown in Table 2, and the equivalent value per unit length of frame is shown in Table 3. Furthermore, the relative rigidity of each

frame is also shown in the last column of the same Table.

Story-rigidity in Two Story Buildings. The following load per unit floor area including both dead and live load is considered, on an average, to participate in the vibration of buildings during earthquakes:

	Light-roofed	Heavy-roofed
Roof level	100 kg/m^2	150 kg/m^2
2nd floor of two story bldg.	180	180

Now consider a two-mass-system which has mass M and rigidity k_2 for the second story, and 1.5 M and k_1 for the first story, respectively. Assuming $2\pi\sqrt{M/k_2} = 0.3$ sec and each fration of critical damping with respect to first or second mode is equal to 0.1, the elastic response of the system can be obtained from the velocity spectrum of an earthquake. If the displacement of each story is calculated by using average velocity spectrum curves[13], and the ratio of the story rigidities which produces equal relative distortions are sought, the relation $k_1/k_2 = 1.75$ will be obtained. This means that in the first floor of a two story building larger quantity of frames should be required than in the second story but it results in larger load carried by unit frame length in the first story. This is not inconsistent with observed results shown in Figs. 7 and 8.

A Proposal on Aseismic Design. From a viewpoint that earthquake force resistance of a wooden building comes from frames with wall panels or bracings and based on the principle to restrict the damage within a small limit, a proposal on earthquake resistant design of wooden buildings is presented here. In this case the coupling effect of the structure and ground should be considered.

In the 1948 Fukui Earthquake, the city situated on thick Alluvium deposit and near epicenter suffered severe shocks, the maximum horizontal acceleration of which was estimated to be over 0.4 g. In the great earthquake of Tokyo in 1923, the maximum horizontal acceleration estimated was 0.2 to 0.25 g at the downtown area on the Alluviam and 0.1 to 0.15 g at the uptown area on the Diluvium. Wooden buildings standing on the Tertiary Bed Rock suffered least damage, generally speaking, as had been observed in the past earthquakes. If the ground condition is classified into four kinds as shown in Table 4, the range of each predominant period is considered to be as shown in the same Table.

The response of wooden buildings to earthquakes depends on the vibrational characteristics of the ground and dynamical properties of the structure. To avoid the quasi-resonance during earthquakes, the natural period of a building with its elongated value should be taken not to coincide with the predominant period of the ground. On the other hand it has been proved that the natural period of a wooden building is mostly governed by the quantity of frames with walls or bracings.[14) 15)]

In the Fukui Earthquake, it is seen, from Fig. 7, that the maximum load carried by unit length of frame which restricted the damage to slight inclination of the building would be 300 kg/m, i.e. about 30 cm/m^2 for a one story building with light roof and about 45 cm/m^2 for one with heavy roof. Based on those values and considerations indicated above, the following method of aseismic design of wooden buildings is proposed:

1) In wooden buildings the quantity of frames with walls or bracings (the length per unit floor area) indicated in item 2) should be provided in a well balance in each principal direction to withstand earthquake force. In this case the effective length of each frame should be calculated by multiplying its real length by the rate shown in Table 3.

The required quantities of frames in wooden buildings are as follows:

a) For buildings on Kind 4 ground; Alluvium, 30 meters or more in thickness, made-land or swampy ground,

	Light-roofed, in cm/m^2	Heavy-roofed, in cm/m^2
One story bldg. or 2nd floor of two story bldg.	30	45
1st floor of two story bldg.	50	75

b) For buildings on Kind 3 ground; Alluvium, less than 30 meters in thickness,

4/5 times the values shown in the Table

c) For buildings on Kind 2 ground; Diluvium,

2/5 times the values shown in the Table

d) For buildings on Kind 1 ground; Tertiary or older strata,

1.5/5 times the values shown in the Table

The natural periods of wooden buildings which have the quantities of frames with walls or bracings prescribed above are considered to be in the range shown in Table 4, where the period during severe earthquakes is assumed to be twice as long. Considering the quasi-resonace effect which amplifies the ground acceleration[16], the earthquake forces to be imposed on structures are generally considered to be in proportional to the quantities of frames prescribed above.

Comment on the Japanese Building Code Requirements. The principle and method of the present Japanese Code Requirements[17] coincide with those presented by the authors in the preceding paragraphs. The required quantity of framings in the Code is generally the same as that proposed by the authors for wooden buildings on firm ground.

However, value required by the Code for buildings standing on soft ground are about 2/3 to 1/2 of those proposed; the former values should be increased up to these values if the damage is to be restricted within a small limit. With respect to the strength rate of each frame, there are some over-estimations on the frames with bracings in the Code, as can be seen in Table 3.

6 CONCLUDING REMARKS

In the present paper, the authors investigated earthquake damage to wooden buildings in connection with the ground conditions and quantities of frames with walls or bracings therein, from which principal lateral force resistance came. On the other hand, static and dynamic tests on such frames of full size were carried out to reveal their seismic behavior. The results thus obtained are as follows:

1) The ground conditions under wooden buildings influence the earthquake damage to a great extent and the softer is the subsoil, the greater is the damage to the buildings. When the thickness of Alluvium under buildings is 30 m or more, significant damage is observed.

2) The earthquake damage to wooden buildings is related with the quantity of frames with walls or bracings provided and the damage increases with the decrease of quantity of frames.

3) From the test results, it is proposed that the angular distortions of frames against earthquake forces be permitted within 1/75 radian, at which they represent the resisting force indicated in Table 3. Furthermore, the prolongation of the natural periods of resisting elements and the variations of damping and absorbing energy in relation with the increase of amplitudes were obtained for each frame as shown in Figs. 20, 21 and 22.

4) The tests proved that the frames with wall panels having horizontal sheathings or diagonal bracings are highly effective in comparison with the frames with Japanese walls of mud plaster over bamboo lath.

Thus, based on the results of investigations and experiments, an aseismic design method for wooden buildings which requires certain quantities of frames with walls or bracings is proposed. The required values vary according to the roof weight, story where the frames are located and conditions of ground on which buildings stand. Also, some comment is given concerning the aseismic regulations on wooden buildings prescribed in the present Japanese Building Code.

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17) Building Standard Law Enforcement Order, Chapter III, Section 3, Article 40-50, Revised, Jan. 1960

NOMENCLATURE

R : Distance from the epicenter in kilo-meters

x : Thickness of Alluvial deposit in meters

y : Damage rate in per cent

P : Horizontal racking load on frame in kg

X : Horizontal distortion of frame in cm

r : Ratio of horizontal racking load to its maximum value

W : Dissipated work in frame during one cycle loading in t.mm

T : Period of frame or building in sec

he : Equivalent fraction of critical damping

FIG. 1

TYPICAL FEATURE OF DAMAGE TO FRAME JOINT

FIG. 2

TYPICAL FEATURE OF DAMAGE TO FRAME JOINT

FIG. 3

TYPICAL FEATURE OF DAMAGE TO WOODEN BUILDING

FIG. 4

TYPICAL FEATURE OF DAMAGE TO WOODEN BUILDING

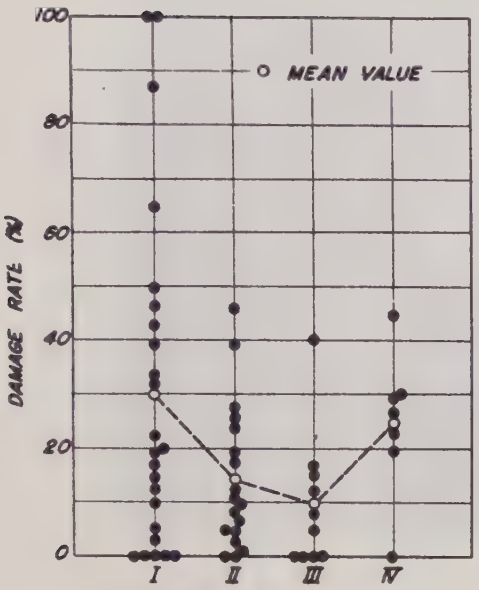

FIG. 5
DAMAGE RATES OF WOODEN BUILDINGS ON DIFFERENT GROUNDS

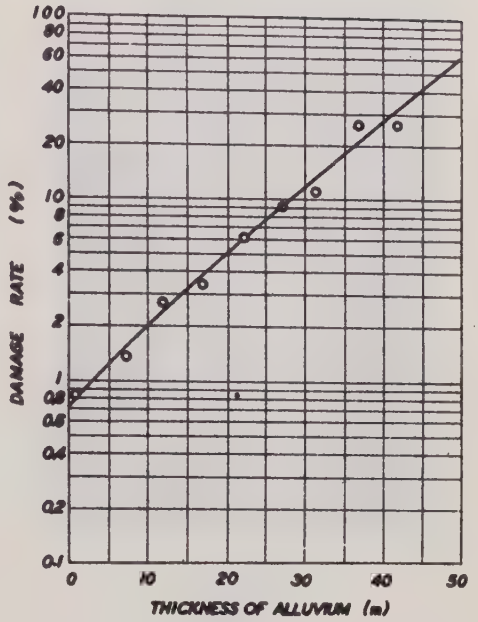

FIG. 6
RELATIONSHIP BETWEEN DAMAGE RATE AND THICKNESS OF ALLUVIAL DEPOSIT

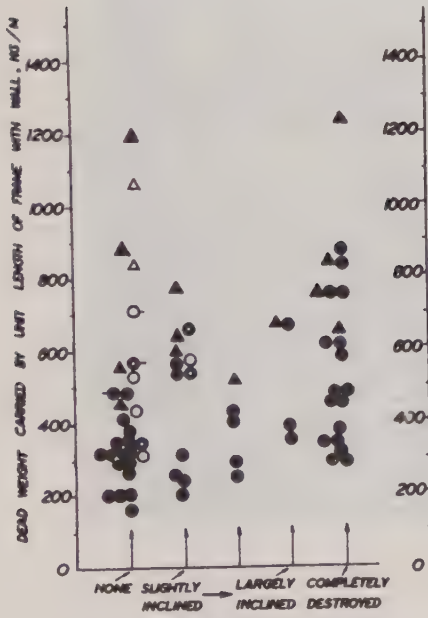

FIG. 7
RELATION BETWEEN DAMAGE AND DEAD WEIGHT CARRIED BY UNIT WALL LENGTH, IN N-S DIRECTION

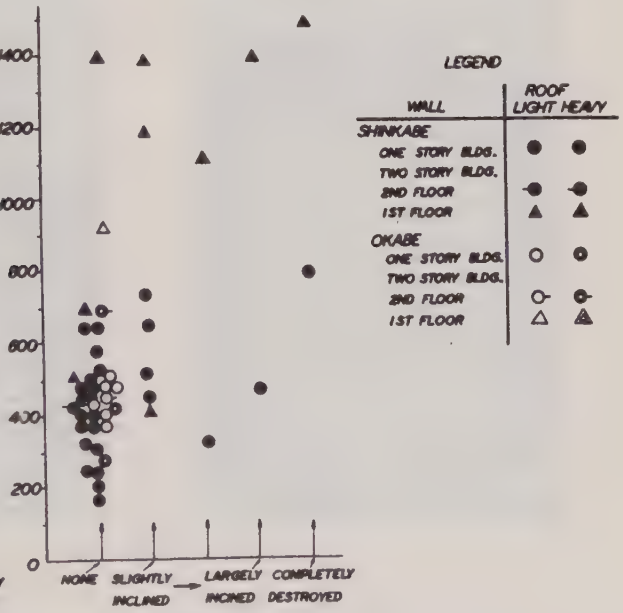

FIG. 8
RELATION BETWEEN DAMAGE AND DEAD WEIGHT CARRIED BY UNIT WALL LENGTH, IN E-W DIRECTION

FIG. 9
TEST ON WALL PANEL SB 3

FIG. 10
TEST ON WALL PANEL OD 3

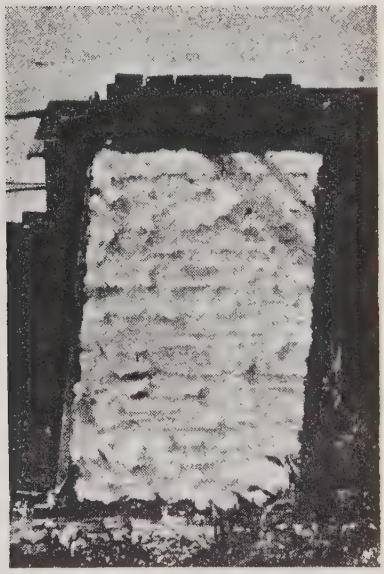

FIG. 11
TEST ON WALL PANEL OB 6

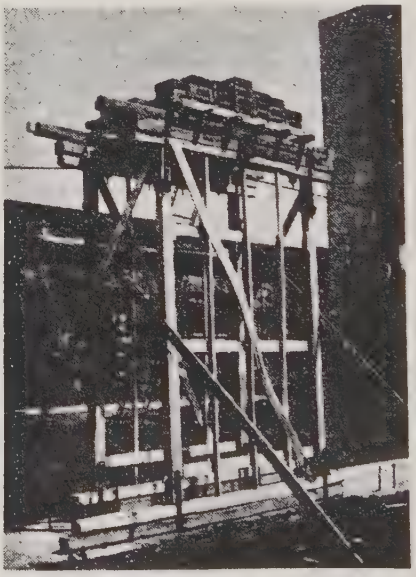

FIG. 12
TEST ON FRAME FWB 3

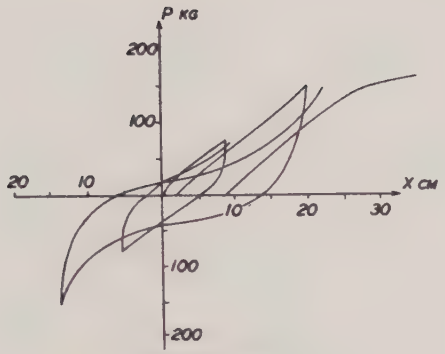

FIG. 13
LOAD - DISTORTION CURVE
FOR WALL PANEL SA 6

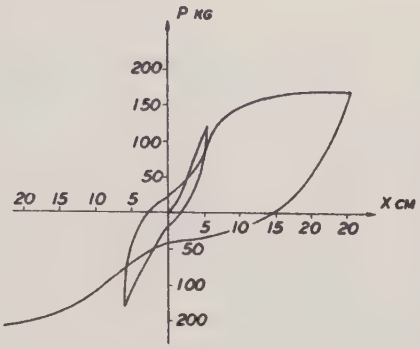

FIG. 14
LOAD - DISTORTION CURVE
FOR WALL PANEL SB 6

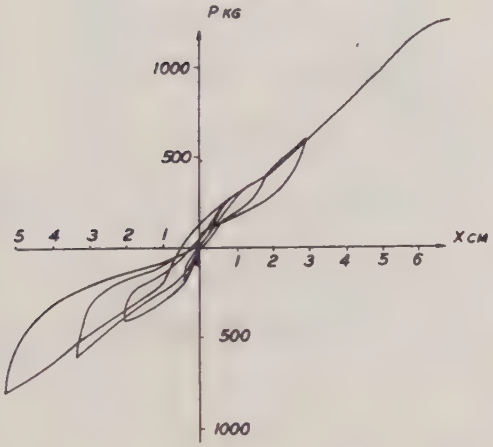

FIG. 15
LOAD-DISTORTION CURVE
FOR WALL PANEL SD 6

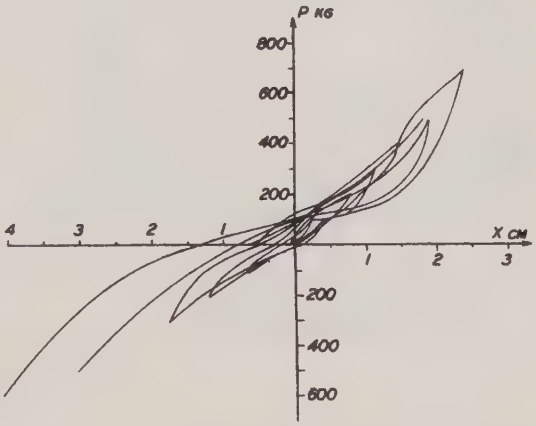

FIG. 16
LOAD - DISTORTION CURVE
FOR WALL PANEL OB 6

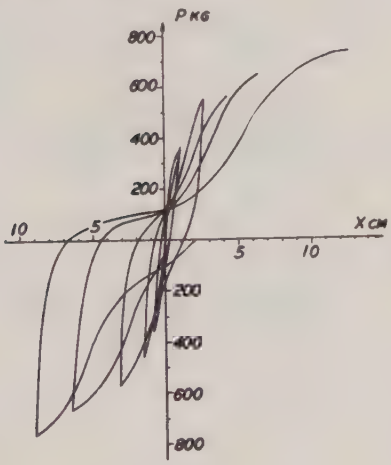

FIG. 17
LOAD - DISTORTION CURVE
FOR WALL PANEL OC12

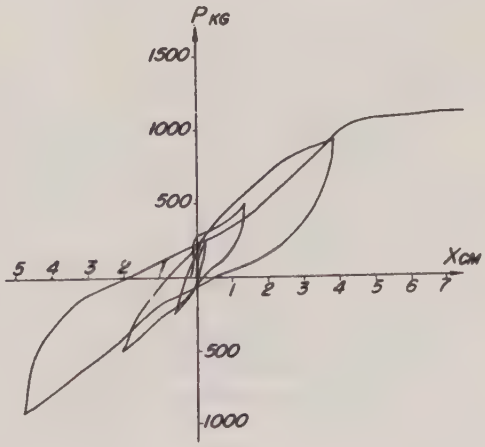

FIG. 18
LOAD-DISTORTION CURVE
FOR FRAME FB1Y2

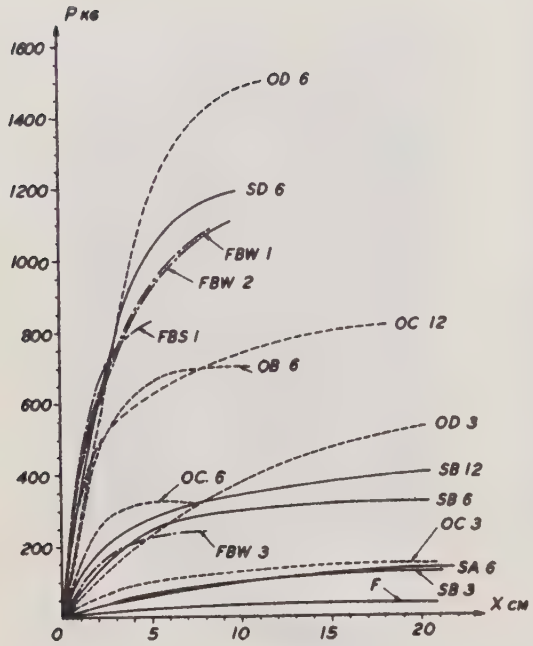

FIG. 19
RELATIONSHIP BETWEEN LOAD AND DISTORTION

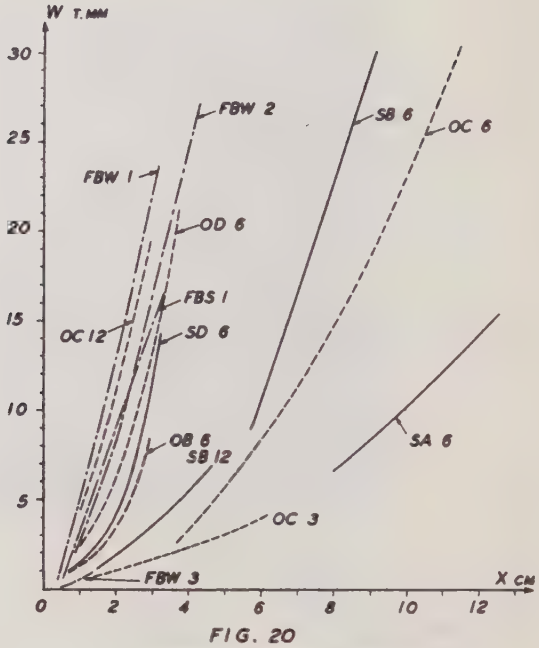

FIG. 20
RELATIONSHIP BETWEEN DISSIPATED WORK AND DISTORTION

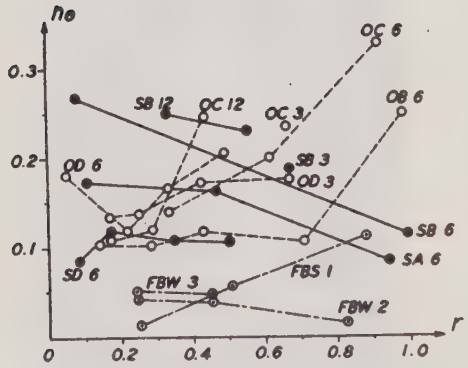

FIG. 22
RELATIONSHIP BETWEEN FRACTION OF CRITICAL DAMPING AND RATIO OF LOAD TO ITS MAXIMUM VALUE

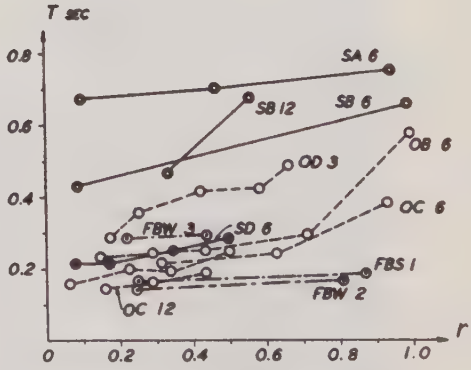

FIG. 21
RELATIONSHIP BETWEEN NATURAL PERIOD AND RATIO OF LOAD TO ITS MAXIMUM VALUE

Table 1

Symbol of frame	Length of frame, in ft	Wall area	Wall finishing		Bracing
			interior	exterior	
SA6	6	above picture rail	mud-plaster	mud-plaster	none
SB6	6	full	mud-plaster	mud-plaster	none
SD6	6	full	mud-plaster	Japanese sheathing	X, 1/5 pillar
OB6	6	full	plaster on wood sheathing	plaster on wood sheathing	none
OC6	6	full	plaster on wood sheathing	wood sheathing	none
OD6	6	full	plaster on wood sheathing	wood sheathing	X, 1/3 of pillar
FBW1	6	frame only	—	—	X, equal in size to pillar
FBW2	6	frame only	—	—	X, 1/2 of pillar
FBW3	6	frame only	—	—	Diagonal, 1/5 of pillar
FBS1	6	frame only	—	—	X,ϕ16 steel bar
F6	6	frame only	—	—	none
SB3	3	full	mud-plaster	mud-plaster	none
OC3	3	full	plaster on wood sheathing	wood sheathing	none
OD3	3	full	plaster on wood sheathing	wood sheathing	X, 1/3 of pillar
SB12	12	full	mud-plaster	mud-plaster	none
OC12	12	full	plaster on wood sheathing	wood sheathing	none

Table 2

Symbol of frame	Length of frame, in ft	Load at 1/75 rad. distortion, in kg	Max. load, in kg
SA6	6	50	160
SB6	6	220	330
SD6	6	920	1200
OB6	6	630	700
OC6	6	320	330
OD6	6	1020	1800
FBW1	6	850	1150
FBW2	6	850	1150
FBW3	6	210	230
FBS1	6	800	800
F6	6	35	40
SB3	3	60	150
OC3	3	90	150
OD3	3	190	600
SB12	12	260	450
OC12	12	600	800

Table 4

Kind of ground	Predominant period of ground, in sec	Natural period of bldg., in sec	
		one story	two story
Kind 1: Tertiary or older strata	0.1 - 0.2	0.35 - 0.7	0.45 - 0.9
Kind 2: Diluvial	0.2 - 0.4	0.3 - 0.6	0.4 - 0.8
Kind 3: Alluvial	0.4 - 0.7	0.2 - 0.4	0.3 - 0.6
Kind 4: Alluvial, 30 meters or more in thickness, made-land	0.4 - 0.6 - 1.0	0.18 - 0.35	0.25 - 0.5

Table 3

	Kind of framing	Load at 1/75 rad. distortion, in kg		Rate
		per 6' length	per 1 m length	
(1)	Frame with "Shinkabe"	220	120	1 (1)
	Frame with "Okabe", sheathed on one side	320	180	1.5 (1.5)
	Frame with "Okabe" sheathed on both sides	630	350	3 (3)
(2)	Frame with a lumber bracing of 1/5 of pillar	210	115	1 (1)
	Frame with X lumber bracing of 1/3 of pillar	700	390	3 (3)
	Frame with X lumber bracings of 1/2 of pillar	850	470	4 (6)
	Frame with X lumber bracings equal in size to pillar	850	470	4 (6)
	Frame with X steel bar bracings having diameter of 16 mm	800	445	4 (6)
(3)	Frame with walls in item (1) and bracings in item (2) combined			Sum of values of the items concerned

Note: Rates in parentheses show the values prescribed in the present Building Code Requirements.

FRENCH REGULATIONS FOR CIVIL ENGINEERING

IN REGIONS BROKEN BY SEISMISM.

Presentation Note

by A. BRENIER

The ORLEANSVILLE catastrophe of 9 September 1954, and the possibility of such destructions in case of further earthquakes in Algeria and possibly in certain regions of metropolitan France or the French Community, have led the Housing and Reconstruction Ministry to preconize particular technical recommendations aiming to make smaller, as far as possible, the effects of earthquakes on the stability of structures.

For that purpose the Housing and Reconstruction Ministry has established a technical commission which has had assemblies on October and November 1954, with M. RAYROLE, General Inspector, being President, and M. SALMON, Chief Engineer, his Assistant, with

- the Bureau Securitas, represented by

  MM. SUQUET, President,
  CAQUOT and LEBELLE, Vice-Presidents,
  LOSSIER and CHAMBAUD, Delegate Members of its Reenforced concrete Commission,
  BRENIER, General Delegate,
  BLEVOT, Assistant to General Delegate.

- the Society for Technical Control and Survey of Construction (SOCOTEC), represented by

  MM. MERCY, Chief of Technical Services,
  HIERHOLTZ, Engineer.

- the Bureau VERITAS, represented by

  MM. BUISSON, Chief of the Building Services,
  GUILLERAULT, Engineer.

(Reporters : MM. BRENIER, BLEVOT and MERCY.).

General Delegate of the Bureau SECURITAS.

Later on, this Commission included other members, especially
MM. BLACHERE, Commissary for ORLEANSVILLE Reconstruction,
GRENOT, Director of the Globe Physics Institute of Algiers,
GOURINARD, Professor of Applied Geology,
and delegates of the Administration and the Constructors.

The map of the regions broken by seismism has been drawn by MM. LAFFITE and GOURINARD, Geologists.

The worked out regulations have been issued on May 1955 under the title : "RECOMMENDATIONS RELATIVE TO CONSTRUCTIONS TO BE BUILT IN REGIONS SUBJECT TO EARTHQUAKES" and are known as "AS 55 RECOMMENDATIONS".

-ooOoo-

French Regulations for protection against earthquake, have naturally taken into consideration the experience acquired in other countries broken by frequent earthquakes. In particular, the principle, which is nowadays universally approved, consisting in substituting to the dynamic calculation a static computation based on the intervention of horizontal forces proportional to masses, has been reconsidered. The allowable stains are considerably increased, and correspond approximately to the limit of elasticity of materials. Yield computing methods may be used.

Considering the circumstances, the values attributed to the seismic coefficients in the AS 55 Recommendations are certainly among the smallest in the world. It has to be explained that they have been fixed in function of several data, among which the seismicity of Algeria, normal regulations concerning constructions independantly from all ideas for protection against earthquakes, and finally the economical aspect of the problem.

France seismicity is weak. That of Algeria is rather well known nowadays thanks to the works of the Globe Physics Institute of Algiers, and professor ROTHE, Director of the Central International Bureau of Seismology of Strasbourg (1) to (6). Algeria is the scene of a rather intense seismic activity and has been struck by several destructive and grave earthquakes, but these have not worn, until now, a character of cataclysm as in certain other parts of the world, in sort that, safe in case of unforeseeable modifications of the seismicity of that region, it seems rather easy to take effective precautions against earthquakes. The correct application of normal regulations for constructions used in France (7) (8) (9) gives to the buildings planned in a simple and rational way, well calculated and well realized non-negligible chances to resist satisfactorily to medium intensity shocks. The rightness of this point of view has been confirmed by the Agadir earthquake of February 29th 1960 ;

with a few rare exceptions, buildings well studied and well realized have remained upright, though they have not been calculated with the hypothesis of an earthquake shock.

The authors of AS 55 Recommendations have consequently insisted on design questions, so as to make future constructions enter in the class of constructions to which normal regulations already confer non-negligible chances of survival. Other prescriptions aim to bring the indispensable complement of stability.

At last, the appreciation of seismic coefficients is part of an economic and human whole, and in this respect, positions may be totally different if we are in a strongly industrialized country or in an under-developped country. In the case of a medium seismicity country like Algeria, which is also not well fitted, industrially speaking, one may think that more human lifes can be saved by building hospitals or struggling against poverty than by increasing the value of antiseismic coefficients.

-ooOoo-

AS 55 Recommendations have been strictly applicated in the whole regions destroyed by the ORLEANSVILLE earthquake of 1954, and under the Administrative Juridiction of the Commissariat for the Reconstruction of Orleansville.

BOUGAINVILLE village, at about 12 miles from ORLEANSVILLE, has been rebuilt according to the AS 55 Regulations, and has undergone two shocks on June 28th and August 15th 1957, the intensities of which, in the International Macroseismic scale, have been respectively VII and VIII. The new buildings have behaved perfectly well.

It is unfortunately deplorable that the application of the antiseismic regulations has worn an imperative character only in the destroyed zone of 1954. Everywhere else, they have remained till now but simple recommendations. We can be but satisfied of the decision taken recently to make them become obligatory by means of a governmental decree.

-ooOoo-

Let us notice finally that these regulations are being reviewed to take into consideration the lessons from the Agadir earthquake and recent acquisitions in earthquake engineering.

-ooOoo-

A. Brenier

RECOMMENDATIONS CONCERNING THE CONSTRUCTIONS TO BE BUILT IN SEISMIC REGIONS.

PREAMBLE

TERRITORIAL ZONES IN ALGERIA

For the whole of the Algerian territory two zones will be distinguished :

- A zone, with low seismicity ;
- B zone, with high seismicity.

These zones are delimited on the annexed map, which is taken from the report of November 28th 1954 by Mr. Laffitte and Mr. Gourinard, professor and Assistant at the Laboratory for Applied Geology of the Algiers Faculty of Sciences.

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1 - GENERAL RECOMMENDATIONS CONCERNING THE DESIGN.

In order to make constructions resist in good conditions to seismic movements such as they manifested themselves till now, without an inacceptable increase of costs, it is recommended to respect the following rules :

1.1 - GENERAL CONCEPTION OF BUILDINGS

1.11 - The height of buildings shall be diminished as much as possible, and especially the ratio of height to width (the width being the smallest lenghth between outer faces of façades); if this ratio exceeds :

- 2.5 for A zone ;
- 2 for B zone ;

special justifications must be furnished.

1.12 - Constructions not well equilibrated in their height or in their inertia are to be avoided.

1.13 - There is to be disposed, as far as possible, an all-over basement, or else, deep massives or reinforced foundations which anchor the construction in the soil.

1.14 - Vaults without tie-rods are to be avoided as well as generally the constructions or parts of constructions the stability of which is incompatible with small movements of supports.

1.15 - Cantilevers, brackets, cornices with important projections and generally all kind of constructional elements not well tied to the framework are to be avoided.

1.16 - In roofings or ceilings avoid the use of not well fixed elements, even if they are of small dimensions.

1.17 - Circulation area is to be disposed in a way to facilitate rapid exit in the case of an earthquake. Each flight of stairs shall form a whole as stiff as possible, well tied to the landing and the framework.

1.2 - FOUNDATIONS

1.21 - Solid ground is to be chosen preferably ; water-bearing soils, fills, falls, alluvions very recent or in thin layers are to be avoided.

1.22 - Deep foundations should be disposed, carefully tied and engaged in the resisting soil, especially in the view of their resistance to lifting forces due to earthquake.

1.23 - All heterogenous foundations are to be avoided.

1.24 - A very resistant tie is to be realized between the foundations and the superstructure.

1.3 - SUPERSTRUCTURE

1.31 - Vertical loads are to be diminished in the upper parts ; the center of gravity of the construction ought to be held as low as possible. Heavy roofing and terraces are to be especially avoided.

1.32 - In framework constructions, joints shall be made stiff. It is most important to make safe the indeformability of the whole structure by efficient wind-bracing in every direction ; for instance by means of cross walls disposed sufficiently near to one another and stiff window basements, these members being tied to one another and to the skeleton by carefully anchored reinforcements.

- The disposition of the reinforcement in framework joints shall allow a correct concreting.

- In columns, the overlaps length of reinforcing toothing bars shall be at least 50 times the diameter of the bars. Overlaps shall be executed without hooks.

1.33 - Efficient connection is to be made between different members of a structure by means of tie-beam or tie-rods (reinforced concrete or steel), horizontally, vertically and diagonally. These ties have to resist to tensile and shear stresses and to stresses resulting from a possible torsion of the whole structure.

- Reinforced concrete constructional members subject to shear shall always be reinforced transversally ; the spacing of that reinforcement shall not exceed the effective depth of the member considered (the depth refered to is the depth from compression face of beam or slab to the center of the longitudinal tensile reinforcement section, as used in bending computations). This does not apply to solid slabs and footing slabs.

1.34 - Special care shall be given to these connections when using precast elements. Proscribe floor constructions with joists running only in one direction without a reinforced concrete slab cast on the spot; precast concrete joists and slab cast on the spot are to be efficiently tied.

1.35 - In masonry construction it is expressly recommended to frame masonry panels by means of horizontal and vertical tie-beams. Spacing of such ties (between parallel members) should normally not exceed 5.00 meters.

- When no such ties are disposed and when masonry is built

in horizontal layers it could be sufficient in some parts of the structures to dispose tie-reinforcements in the masonry joints, on condition that such reinforcement is carefully anchored in the vertical skeleton members or in orthogonal walls. Such reinforcements are to be placed in deep horizontal joints (3 to 4 cm deep), put at most every 50 cm. Cross-sectional area of such reinforcement is to be about 1 cm^2 in each joint.

- Masonry is to be pugged with ciment mortar, made with clean sand not containing agregates the size of which is inferior to 0.4 mm. Masonry materials should be abundantly watered just before building them in.

- Isolated pillars and masonry walls between windows of small depth shall be avoided.

1.36 - Framings tied to the skeleton or the tie-beams are to be disposed around openings.

1.37 - Special precautions are to be taken for corner buildings (especially as to wind-bracing) and for transformations or heightenings where stability of both old and new structure has to be proved.

1.38 - It shall be avoided to transmit to small surfaces important efforts due to accelerations both horizontal and vertical ("punching" or "ramming" effects at the end of trusses, beams,...)

- Elastic tampons are to be disposed in expansion joints between blocks of same inertia or nearly same inertia. On the contrary, large joints are to be disposed between blocks of markedly different inertia.

1.39 - Not-fragile gas and water piping shall be used, as well as carefully protected electric lines.

1.40 -When planning the execution of works, it is to be avoided to have important breaks between the execution of the skeleton or bearing walls and that of front wall fillings and inner partitions which contribute efficiently to the wind-bracing of the structure.

2 - COMPUTATION RULES.

Stresses shall first be calculated under the effect of dead loads, live loads and climatic loads according to regulations in vigour at the time of the construction[+]. Then earthquake

[+] Regulations to be applied at present are the codes edicted by the Ministry of Housing and Reconstruction and known as : Règles BA 1945 for reinforced concrete, CM 1946 for steel structures and NV 1946 for the computation of effects of wind and snow.

effects shall be calculated as indicated in the following :

2.1 - Earthquake forces.

The inertia forces, due to seismic movements of the ground, which develop in the structure, may act in any direction. It will be sufficient to consider simultaneously or successively the effects of a horizontal and vertical component, these components being defined as follows :

2.11 - Horizontal component.

For a determined element of the construction, this component acting in any horizontal direction and applied to the center of gravity of the considered element, will be σ P.

σ is a "seismic coefficient", equal to the product $\sigma_1 \sigma_2 \sigma_3$ (see annexed tables).

- σ_1 is a "zone coefficient" which, up to 10 m. height above ground level, will equal

0.035 in A zone (1)

0.070 in B zone (1)

Over 10 m. height above ground level, the values of σ_1 as given above shall be increased with 2% per meter of height.

Taking for an example 16.0 m. height, we will have :
0.070 (1+ 0.02 x 6) = 0.0784 in B zone.

- σ_2 is a "ground coefficient" wich depends on foundation conditions. This coefficient which will equal 1 in usual foundation cases, can vary between two extreme values :
0.75 for constructions founded on rock,
1.25 for constructions founded on loose and water bearing soils.

---

(1) - These values correspond to the following base coefficient

0.05 in A zone
0.10 in B zone

multiplied by a reducing factor of 0.7. This reducing factor accounts for the fact that the seismic effects the intensity of which varies very rapidly with time, will be assimilated to statical forces liable to act during a considerable time.

- σ_3 is a "foundation depth coefficient" which will equal 1 when the construction has a over-all basement or deep massive reinforced foundations, and 1.25 in other cases.

P will be equal to :

1) dead loads only, corresponding to the member considered, for housing buildings.

2) dead loads plus half the live loads acting direcly on the member considered, for stores, warehouses and industrial buildings.

3) dead loads plus total live loads acting directly on the member considered, for reservoirs, tanks and silos.

2.12 - Vertical component :

This component will equal $\pm$ 2 σP, σ and P being calculated conforming to paragraphe 2.11 and being agreed that for σ_1 the value fixed above for height up to 10 metres, will be taken without any increase for greater height.

2.13 - In case of constructions having floors, computations will be made by applying the horizontal forces at each floor level, and by accounting in the evaluation of P dead loads and possibly the fraction of live loads defined above, corresponding to the floor presently considered.

2.14 - For isolated constructions (factory chimneys, tanks, fence walls, etc.) the seismic coefficient defined above should be doubled.

2.15 - For chimneys (beginning from last floor) and for structural elements jutting out from front or side walls of buildings (corbels, projecting elements of balconies, cornices) the seismic coefficient defined above shall be multiplied by 3 ; for balconies, verification shall be made for the parapet and for the whole balcony.

2.2 - Allowable unit stresses :

Stresses shall be calculated under the simultaneous action of :

- dead loads,
- service live loads,
- earthquake,
  excluding climatic loads (wind, snow, temperature).

2.21 - In certain verifications it could be more defavorable to assume that live loads or some of them are null. One shall equally account for the fact that inertia forces of seismic origin can act upwards.

2.22 - It has to be proved for each member that the above defined effects on construction or part of it are inferior to ultimate strength capacity. This verification has to be made by means of an ultimate strength design method based on experiments of proved evidence.

Provisionally, for lack of an official method according to regulations for ultimate strength design, the classical methods, may be used i. e. those for elastic range.

In this case computations will be made by taking the following values for unit stresses allowable under these conditions :

- for the steel or steel constructions, or concrete reinforcements : the conventional limit of elasticity.

- for the concrete of flexural members of reinforced concrete skeletons : 8/10 of compressive strength, this being measured at 90 days of age on cubical specimens of 14.1 or 20 cm side length (tensile strength of concrete will be considered as null) ;

- for the concrete members of reinforced concrete skeletons, the section of the members being under compression on their whole area : 6/10 of compressive strength determined under above conditions ;

- for masonry and plain concrete : the triple of unit stress usually allowable ;

- for soils if they are plain rock : the triple of unit stress usually allowed ;

- for loose and water bearing soils : the unit stress usually allowable ;

- for soils that cannot be classified in the above two categories the double of unit stress usually allowable.

2.23 - Filling panels can be accounted for in computations relative to the stability of constructions on condition that those panels are wholly framed by skeleton members or are made of masonry treated as indicated in 1.35.

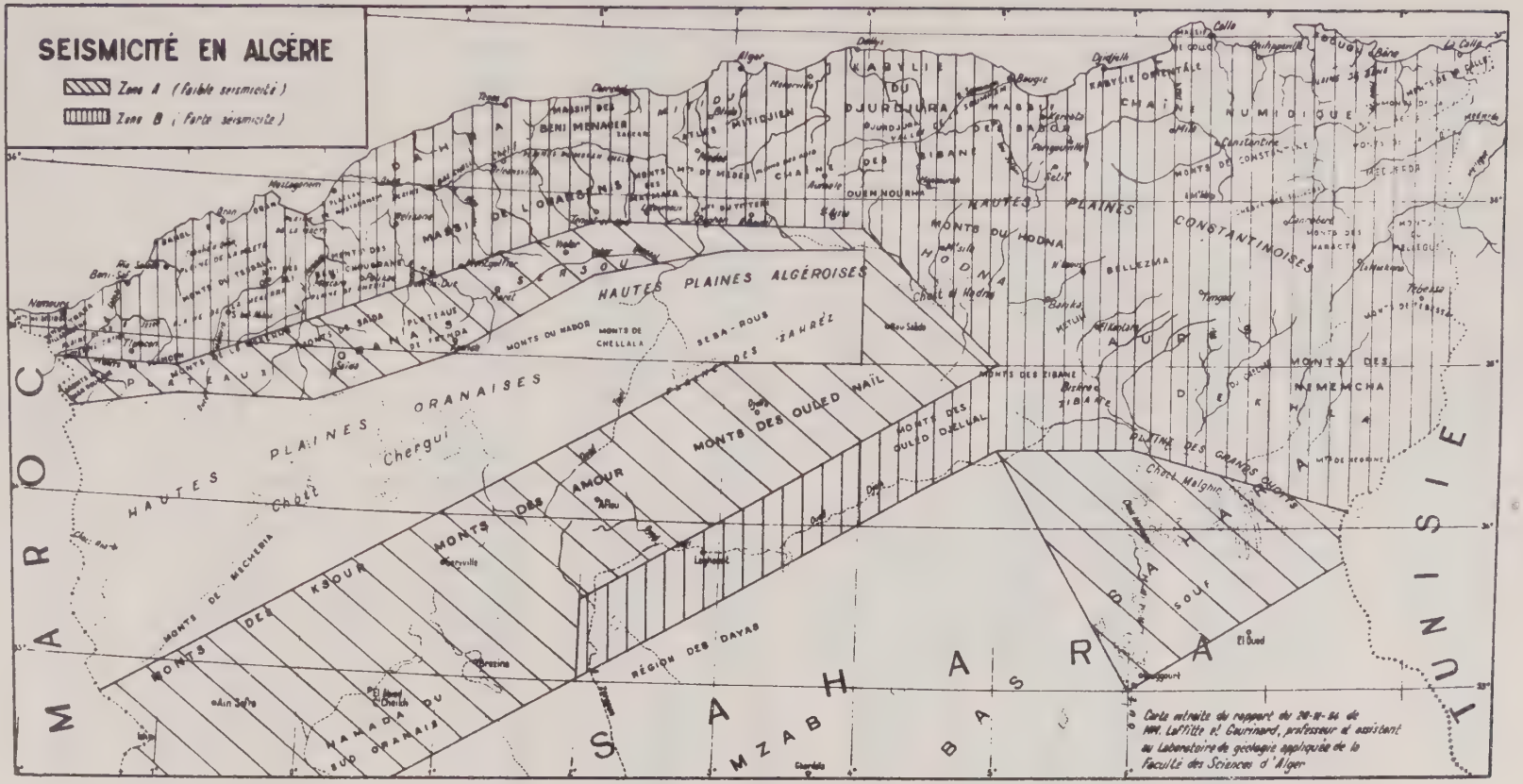
SEISMIC COEFFICIENTS TO BE ACCOUNTED FOR IN STATICAL COMPUTATIONS OF CONSTRUCTIONS TO BE BUILT IN EARTHQUAKE ZONES

	Heigth above ground level	Soils of average consistency		Rock soils		Loose and water bearing soils	
		Deep foundations	Superficial foundations	Deep foundations	Superficial foundations	Deep foundations	Superficial foundations
Horizontally	up to 10,00 m	0,0350	0,0437	0,0262	0,0328	0,0437	0,0547
	to 20,00 m	0,0420	0,0525	0,0315	0,0394	0,0525	0,0656
	to 30,00 m	0,0490	0,0612	0,0367	0,0459	0,0612	0,0765
	to 40,00 m	0,0560	0,0700	0,0420	0,0525	0,0700	0,0875
Vertically	any heigth	0,0700	0,0875	0,0525	0,0656	0,0875	0,1094

SEISMIC COEFFICIENTS TO BE ACCOUNTED FOR IN STATICAL COMPUTATIONS OF CONSTRUCTIONS TO BE BUILT IN EARTHQUAKE ZONES

	Height above ground level	Soils of average consistency		Rock soils		Loose and water bearing soils	
		Deep foundations	Superficial foundations	Deep foundations	Superficial foundations	Deep foundations	Superficial foundations
Horizontally	up to 10,00 m	0,0700	0,0875	0,0525	0,0656	0,0875	0,1094
	to 20,00 m	0,0840	0,1050	0,0630	0,0787	0,1050	0,1312
	to 30,00 m	0,0980	0,1225	0,0735	0,0919	0,1225	0,1531
	to 40,00 m	0,1120	0,1400	0,0840	0,1050	0,1400	0,1750
Vertically	any height	0,1400	0,1750	0,1050	0,1312	0,1750	0,2188

SEISMICITY IN ALGERIA A zone (low seismicity) B zone (high seismicity)



Map taken from report dated 28.11.54 by Mr. Laffitte and Mr. Gourinard, Professor and Assistant at the Laboratory for Applied Geology, at the Algiers Faculty of Sciences.

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ANALYSIS OF THE EARTHQUAKE RESISTANCE OF FRAME BUILDINGS TAKING INTO CONSIDERATION THE CARRYING CAPACITY OF THE FILLING MASONRY

S.Sachanski[1/]

SUMMARY

A method is elaborated for the analysis of the earthquake resistance of multistorey buildings, taking into consideration the carrying capacity of the filling masonry.

The results are given of certain theoretical and experimental investigations of a section of a building consisting of brick masonry and a reinforced concrete frame. The carrying and deformative capacities of the masonry subjected to horizontal forces are determined. Formulae are given for the stiffness of the masonry and the distribution of the horizontal force between the frame and the masonry. The tables and pictures illustrate some of the results of the tests of walls with smaller dimensions, of full-size elements, as well as walls from old frame buildings. The results of the investigations may also be used for the earthquake analysis of frame buildings with other kinds of filling masonry or light concrete.

Frame buildings consist of a reinforced concrete frame, which takes over the vertical loading and of filling masonry which serves as isolating partition. When they are subjected to horizontal forces the frame and the masonry work jointly. This common action is determined by the small deformative capacity of the masonry. On account of this the masonry may be used for taking over the greater part of the horizontal forces, provided the dimensioning is correct.

Some of the results of the investigations on the joint work of the framework and the masonry subjected to the action of horizontal forces are presented in this report.

We assume that the size of the earthquake forces is determined according to the existing standards.

The analysis of the space frame and the masonry under the action of these forces is a difficult task; on account of this some authors consider buildings and other structures as consisting of separate diaphragms (fig.1). It is necessary that the whole horizontal force be distributed among the separate diaphragms, before the analysis of each diaphragm.

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Several methods have been worked out for the distribution of the horizontal loading among the diaphragms. A brief and simple method is that of Streletzki /1/.

This method is particularly suitable for monolithic frame buildings where the floor slabs have a high stiffness and during their horizontal displacement act as undeformable disks and distribute the horizontal loading in proportion to the stiffness of the walls beneath. The term stiffness of walls and frames denotes the force causing a unit displacement in the respective element.

If the resultant of the external forces R is symmetrical in respect to the cross section of the building (i.e. passes through the center of gravity) and if the discs are symmetrical and of equal stiffness, then the floor slab will undergo a translatory displacement and the force in any diaphragm will be determined by /1/

$$H_k' = R_H \frac{K_k}{\Sigma K_i} \qquad /1/$$

where K_k is the stiffness of the system of discs along a certain axis.

In most cases, however, the diaphragms are not symmetrically situated and have different stiffness; then the floor slab will not only undergo a translatory displacement but will also revolve around its center of rotation, which will bring about a redistribution of the forces.

In order to determine the rotation centre of the floor slab, the stiffness of the individual discs have to be defined

K_i^x - for diaphragms parallel to the x axis
K_i^y - for diaphragms parallel to the y axis

Then the coordinates of the rotation center are defined with

$$x_c = \frac{\Sigma K_i^y x_i}{\Sigma K_i^y} \qquad y_c = \frac{\Sigma K_i^x y_i}{\Sigma K_i^x} \qquad /2/$$

The coordinates of the rotation center may also be determined graphically by means of a polygon of forces and a funicular polygon, taking the stiffness of the different diaphragms as forces. The intersection of the resultants of the stiffnesses in the direction of the two axes of the structure defines the rotation center.

Expressing every stiffness through the smallest of them

$$K_n = d_n K_1 \qquad d_n = \frac{K_n}{K_1} \qquad /3/$$

the additional effort due to the rotation is

$$H_k'' = \frac{R_H \cdot \ell \cdot d_k a_k}{\Sigma d_i^2 a_i^2} \qquad /4/$$

where ℓ = excentricity of R with respect to the rotation center

a_x= distance between the respective diaphragm and the rotation center.

Then the entire force corresponding to a certain disc, due to translatory displacement and rotation, is:

$$H_x = H_x' + H_x'' \qquad /5/$$

After the force is distributed among the separate frames with masonry (diaphragms) in proportion to their stiffness, the following fundamental problems should be resolved: 1) what proportions of the force are taken over by the masonry and by the frame; 2) how to determine the stiffness of the masonry and of the frame; 3) how to calculate the masonry so that it can take over the corresponding force. These main questions, related to certain secondary ones, are solved through theoretical, photoelastic, model and full scale investigations. The latter have been carried out simultaneously and the results compared, supplemented and specified. The theoretical investigations and the full size tests of elements like the one shown on fig.1 have encountered considerable difficulties. On account of this the investigations have been carried out with elements consisting of one masonry cell surrounded by a reinforced concrete frame (fig.3). The results of the photoelastic tests determine the examination of the model of fig.3 instead of that of fig.1[1].

I. THEORETICAL INVESTIGATIONS

The structure on fig.3 consists of two heterogeneous materials - masonry and frame; because of that the stresses arising from the horizontal force H are considered separately for the masonry and the frame, taking into account the joint work of the two elements. For this purpose the stresses arising from the interaction between the frame and the masonry have been examined.

This problem of interaction (contact problem) has been solved with the aid of the method of redundant reactions in the following manner[2]:

1. The bond between masonry and reinforced concrete frame is substituted by joints, which take over the and τ efforts (fig.3).

1/ The results of the photoelastic tests are not presented here, since the alloted space is not sufficient for the photographs which should be appended.

2/ Experiments have been carried out to define the stresses in the masonry and the contact zone with the aid of other methods: with the method of the finite differences with the extension of the stress function over the whole disc; with a polynome of n-power in the contact zone; with orthogonal cross sections and the introduction of unknown efforts and so on.

2. The joints are cut and their action is substituted by the action of the redundant reactions - 30 in all.

3. A system of equations with 30 unknown quantities is worked out, which expresses the condition, that at the place of the cross section the sum of the mutual displacements is equal to zero.

4. The coefficients in front of the unknown quantities represent the sum of the individual displacements in the masonry and the frame.

The displacements in the masonry due to the unit loadings are defined through the integration of the stresses in the masonry, which is considered as a wall beam. The stresses are calculated after the method of the finite differences (network method). In the most common case of unit loading the function of the stresses is determined by solving a system of equations with 15 unknown quantities. In order to simplify the computation the unit loadings are broken up into symmetrical and inversely symmetrical. The deformed state of the masonry is determined with the aid of the Williott diagram.

The displacements thus determined in the wall beam and the frame are substituted in the system of equations with 30 unknown quantities, which is solved through one of the familiar methods. The unknown efforts which are thus found define the character of the stresses in the contact zone (fig.4).

II. DETERMINATION OF THE STRESSES IN THE MASONRY

In theoretical investigations the masonry is considered as an elastic element. This is done in order to use the methods of the theory of elasticity, on the one hand, and to give a solution, which may be used with filling masonry made up of different materials, on the other. The corrections taking into consideration the peculiarities of a given material are made after carrying out a series of tests for finding out to what extent the theoretical conclusions coincide with the experiments. If necessary the theoretical conclusions are corrected in accordance with the peculiarities of the material.

The solution of the problem of interaction (contact problem) indicates the way in which the horizontal force H is transmitted to the masonry.

After a series of examinations the conclusion is drawn that the law of the transfer of the loading from the frame to the masonry should be conceived as a stress function, which allows the direct determination of the stresses in the masonry.

Such a stress function has the form

$$\varphi = \varphi_1 + \varphi_2 + \varphi_3 + \varphi_4 \qquad (6)$$

where

$$\varphi_1 = \sigma_\kappa \left[\frac{xy}{6\alpha\beta} (y^2\beta^2 + x^2) - \frac{1}{2} xy\beta + (x^2 + y^2\beta^2) \right]$$

$$\varphi_2 = 0.5\sigma_\kappa \left[\left(\frac{2a}{\pi}\right)^2 \cos\frac{\pi}{2}\xi - \frac{0.345(x^2-a^2)(y^2-\beta^2)^2}{a^2(5.25a^4 + 3a^2\beta^2 + 5.25\beta^4)} \right]$$

$$\varphi_3 = 0.5\beta\sigma_\kappa \left[\left(\frac{2\beta}{\pi}\right)^2 \cos\frac{\pi}{2}\eta - \frac{0.345(x^2-a^2)^2(y^2-\beta^2)}{\beta^2(5.25a^4 + 3a^2\beta^2 + 5.25\beta^4)} \right]$$

$$\varphi_4 = K\sigma_\kappa \left[\frac{xy}{6\alpha\beta} (y^2\beta^2 + x^2) - \frac{1}{2} xy\beta - \frac{1}{2}(x^2 + \beta^2 y^2) \right]$$

$$\beta = \frac{a}{b} \qquad \xi = \frac{x}{a} \qquad \eta = \frac{y}{b} \qquad W_1 = \frac{1.38(1+\beta^4)}{5.25 + 3\beta^2 + 5.25\beta^4}$$

φ_1 and φ_4 are polynoms, whereas φ_2 and φ_3 are defined after the variation method. The function φ gives the stresses along the contour of the wall beam which are similar to the ones obtained from the solution of the contact problem as well as the stresses in the wall beam (fig.4).

$$\sigma_y = \sigma_\kappa \left\{ \xi\eta(1+\kappa) - 0.5 \left[\cos\frac{\pi}{2}\xi + (3\xi^2 - 1)(\eta^2 - 1)^2 W_1 \right] + 1 - \kappa \right\}$$

$$\sigma_x = \sigma_\kappa \beta^2 \left\{ \xi\eta(1+\kappa) - 0.5 \left[\cos\frac{\pi}{2}\eta + (\xi^2 - 1)^2(3\eta^2 - 1) W_1 \right] + 1 - \kappa \right\} \qquad /7/$$

$$\tau_{xy} = -\sigma_\kappa \beta \left\{ \left(\frac{\xi^2 + \eta^2}{2} - \frac{1}{2} \right)(1+\kappa) - 2\xi\eta(\xi^2 - 1)(\eta^2 - 1) W_1 \right\}$$

The contour stress σ_κ is defined by the condition of equilibrium of external and internal (contact) forces.

$$\sigma_\kappa = \frac{N}{1.46 a d \beta} \qquad /8/$$

III. DETERMINATION OF THE CARRYING CAPACITY OF THE MASONRY

Taking into consideration that during the tests the first fissure (i.e. the ultimate state) appears in the middle of the masonry in the direction of the compression diagonal, we come to the conclusion that destruction is due to the principal tensile stresses.

$$\sigma_p = \frac{\sigma_x + \sigma_y}{2} \pm \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2} \qquad /9/$$

Therefore the ultimate state on the basis of which the carrying capacity of the masonry has to be determined is

$$\sigma_p \leq R_o \qquad /10/$$

for the middle of the wall beam.

By putting x = 0, y = 0 in (7) and substituting in (9) and (10) the results obtained for the stresses, and expressing σ_κ through (8), we obtain the force which causes the first diagonal fissure in the masonry

$$N_m = \frac{3ld(\beta - 0.5\beta^2)}{F} R_o \qquad /11/$$

valid for $\beta < 2.5$

where $F = 1 + \beta^2 - \sqrt{\beta^4 - 0.8\beta^2 + 1}$

l and d length and thickness of the masonry

The use of this method for the determination of the carrying capacity of a masonry with openings encounters considerable mathematical difficulties and the expressions obtained are rather long and unfit for practical use. On account of this the carrying capacity of a masonry with openings is determined with the aid of that without openings by introducing a "factor of the opening" k.

$$N_{m.o} = k_1 \frac{3ld(\beta - 0.3\beta^2)}{F} R_o \quad /12/ \qquad k_1 = \left(1 - \frac{2a_1}{5l} - \frac{3}{5}\frac{b_1}{h}\right)$$

where l = length of the masonry, a_1 length of the opening
d = thickness,
R_o = tensile strength.

When there is no opening ($a_1 = o$, $b_1 = o$) from (12) we obtain (11). When there is no masonry ($a_1 = l$, $b_1 = h$) we obtain N=0.

IV. DETERMINATION OF THE DEFORMATIONS OF THE MASONRY

On the basis of the stresses obtained (7) for the coordinate system xy (fig.4) the stresses for x^1y^1 are determined as well.

$$\sigma_{y'} = \frac{a^2}{a^2+b^2}\sigma_x + \frac{b^2}{a^2+b^2}\sigma_y + \frac{2ab}{a^2+b^2}\tau_{xy} \quad /13/$$

The specific shortening of the compression diagonal is expressed with

$$\varepsilon_s = \frac{2}{l}\int_0^{h/2} \frac{\sigma_{y'}^2}{\gamma E}\, dy' \quad /14/$$

It is assumed that the stresses from x=0, y=0 to x=a, y=b change lineary

$$\sigma_y = \sigma_{y_o} + l\,\frac{\sigma_{xa}b - \sigma_{xo}}{l}\,y \quad /15/$$

After determining σ_y from (15) and (13), and from (8) and substituting them in (14) we obtain

$$\varepsilon_s = \frac{0.2k^2N^2C}{E_m d^2(k^2+l^2)^2} \quad /16/ \text{ where } C = 5.2\left(\beta^4 + \frac{1}{\beta^4}\right) - 7.66\left(\beta^2 + \frac{1}{\beta^2}\right) + 11.76$$

The "factor of the opening" k_1 is introduced for masonry with openings.

$$\varepsilon_{s.o.} = k_1 \frac{0.2k^2N^2C}{E_m d^2(k^2+l^2)^2} \quad /16a/$$

V. DETERMINATION OF THE STIFFNESS OF THE MASONRY AND THE DISC

The horizontal displacement in the masonry is determined on the basis of the specific deformation in the compression diagonal (fig.5).

$$\Delta h = \frac{k^2 C N^2}{l(l^2+k^2)d^2\gamma E_m} \quad /17/$$

Proceeding from the definition of the stiffness of the masonry (the force causing a unit displacement) we obtain

$$K_m = \frac{2 \cdot 2d}{h} \sqrt{\frac{2(l^2+h^2)}{c} \gamma E_m} = d \sqrt{\frac{BE_m}{c}} \qquad /18/$$

$$K_{m0} = K_1 d \sqrt{\frac{BE_m}{c}} \qquad /18^a/$$

The stiffness of the reinforced concrete frame in a given floor can be determined through (19), deduced on the condition, that the beams of the frame together with the slabs have an infinitely great moment of inertia

$$K_s = \frac{12 E_c}{l^3} \Sigma J_x \qquad /19/$$

Then the stiffness of the system of diaphragms in a given floor is

$$K_d = K_s + \Sigma K_m \qquad /20/$$

VI. DISTRIBUTION OF THE FORCE BETWEEN THE FRAME AND THE MASONRY

The whole force which the disc has to take up is to be distributed between the frame and the masonry according to their stiffness. On the basis of the condition, that the deformations of the masonry and those of the frame are equal an expression is obtained which determines the part of the force taken over by the masonry (21)

$$N_m = \sqrt{A^2 - 2AH} - A \qquad /21/ \qquad A = \frac{K_m}{2K_s} l$$

There remains for the frame

$$N_s = H - N_m$$

With the aid of the above formulae the analysis of the earthquake resistance of a frame building can be carried out in the following sequence: 1) The stiffnesses of the discs situated along the respective axes of the building are determined by means of (20), (19), (18) and (18^0); 2) The rotation center of the building is determined through (2); 3) The horizontal force is distributed among the discs in a given floor by means of (1) or (5); 4) The force, which corresponds to a system of discs is distributed between the frame and the masonry by means of (21); 5) Through (11) it is verified whether the masonry is able to take over its share of the load; 6) The reinforced concrete frame is dimensioned with the familiar methods after determining the size of the forces in the remaining floors as well.

In order to solve these problems 26 diaphragms of different characteristics and dimensions were tested (table 1). The tests were carried out with a static horizontal force (as on fig.3).

The following more essential conclusions were drawn from the testing of the discs:

1. When loading the disc with a horizontal force one of the diagonals is subjected to compression and the other to tension. As a result of the tension the first fissure in the masonry without opening appears in the middle of the disc in the direction of the compression diagonal (fig. 6). The increase of the loading leads to an increase of the fissures or to the appearance of other fissures parallel to the first one. The further increase of the loading leads to the occurrence of fissures in the concrete frame as well - first on the internal and then on the external side of the reinforced concrete frame at a distance of 1/5 to 1/2 l or h measured from the non-compression corner. The sequence of the occurrence of fissures in the discs with opening is seen on fig.7.

2. The deformations which give rise to diagonal fissures are almost identical for masonries made with mortars of different strength.

3. The deformation of the masonry is small until the appearance of the diagonal fissure, on account of which the reinforced concrete frame takes up a smaller part of the horizontal force (5 - 20%).

4. The nature of the deformations in the contact zone corresponds to the theoretically defined law.

5. The carrying capacities of the discs turned out to be 20-30% higher than the ones calculated according to (11).

6. The stiffness of the masonry depends on the type of bricks as well as on the composition and strength of the mortar.

7. The openings of the masonry decrease substantially its carrying capacity (2-5 times and more).

VII. FULL SCALE INVESTIGATIONS

The full scale investigations were designed to check the results of the theoretical and model analysis of full-size discs.

The first series of full scale tests were carried out with the walls of an old five-storey frame building with a statical horizontal force. Three 3,5 x 3,0 m walls (fig.8) were tested, two of which 30 cm thick and the third - 15 cm.

The second series of full scale tests were made on brick walls with usual dimensions (table 2, fig.9).

The third series of tests were performed with discs of brick masonry with openings (fig.10).

The fourth series of tests were carried out with discs of lightweight concrete filling with or without openings (fig.11).

Some of the results of the tests are presented in table 2. The following inferences are made from the full-size tests:

1. The nature of the deformations and the fissures is the same as in the model investigations.

2. The carrying capacity determined by way of full-size tests is close to the one determined by calculation with (11).

3. The specific carrying capacity (for a unit cross section) of the full-size masonry is lower than the specific carrying capacity of the masonry obtained through model investigations. This fact can be explained by the different scale modulus of the elements tested in both cases. Therefrom the conclusion can be drawn, that the model investigations cannot be used for full-size elements without taking into consideration the scale modulus.

CONCLUSION

The results of some of our investigations represent an attempt to give a more exact solution of a problem notable for its complexity.

The theoretical investigations were carried out without taking account of the kind of filling mateiral. This makes it possible to use the results for every kind of material, provided certain corrections taking account of the properties of the material are made. This is also confirmed by the fact that the discs[1/] tested with a filling of monolithic slag concrete do not differ substantially from the ones with brick masonry. It is only necessary to introduce a correction factor. In this way the expressions obtained may also be used for the analysis of the earthquake resistance of certain types of prefabricated houses.

1/ The results of these investigations are not given here.

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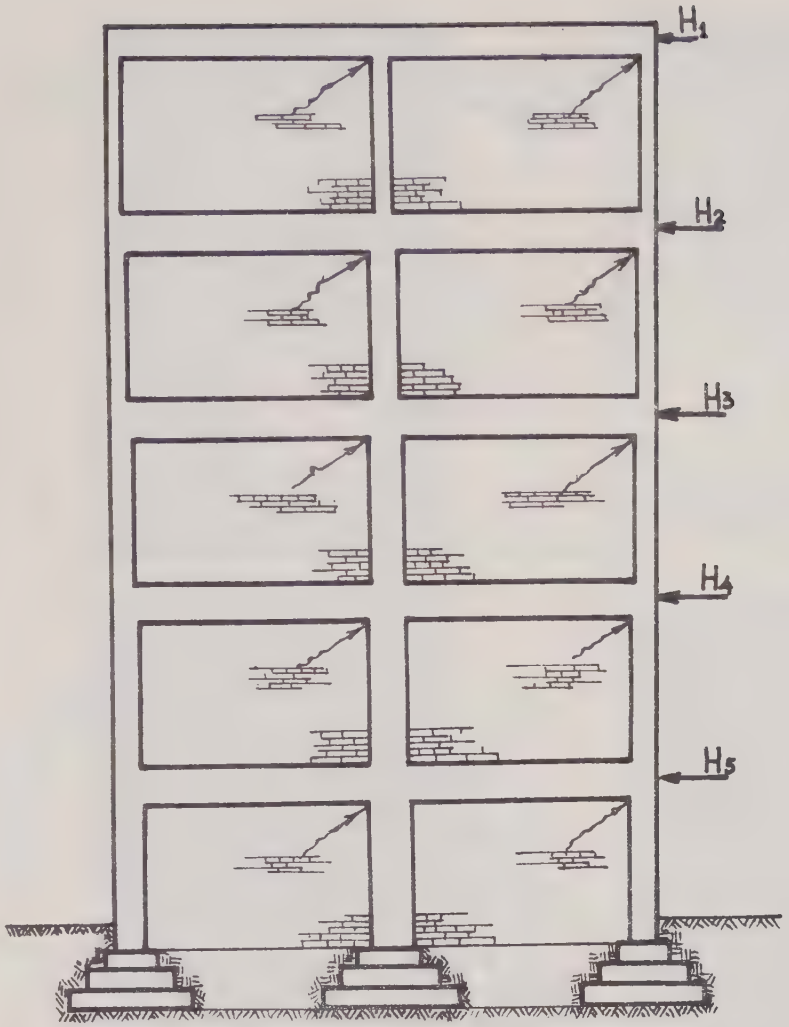


Fig.1
Transverse diaphragm in the frame building

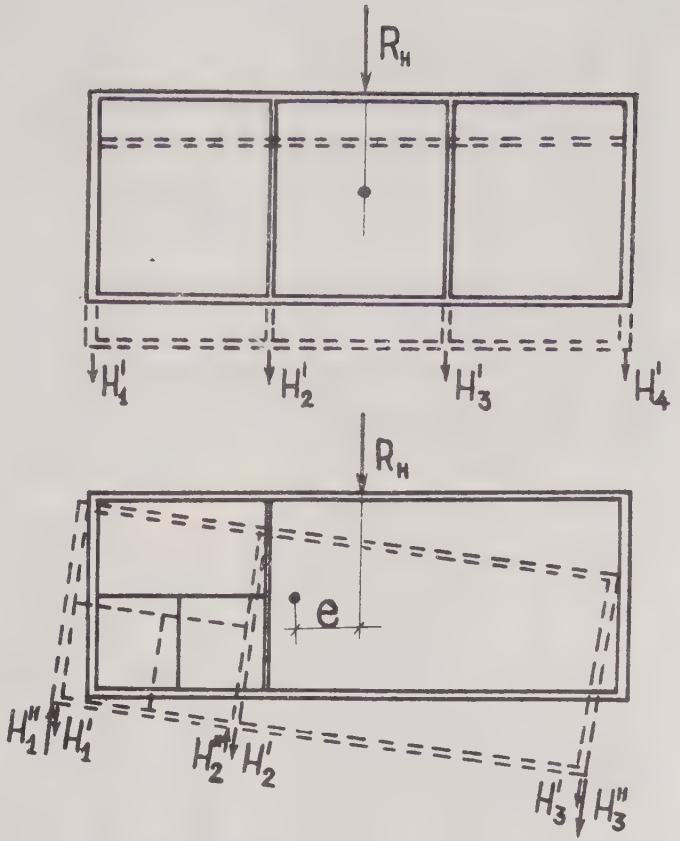


Fig.2
Displacement of the frame building under the action of horizontal forces; a - by symmetrical stiffness; b - by asymmetrical stiffness.

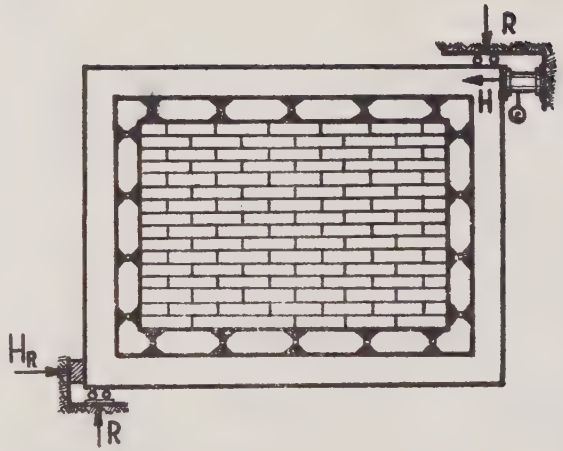


Fig.3 Schematic joint between frame and masonry. Pattern of loading

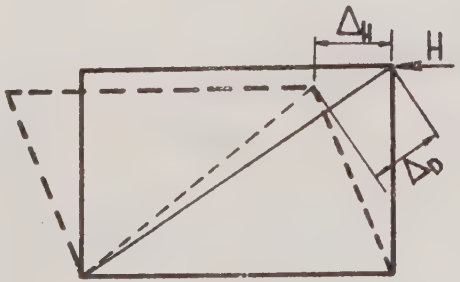
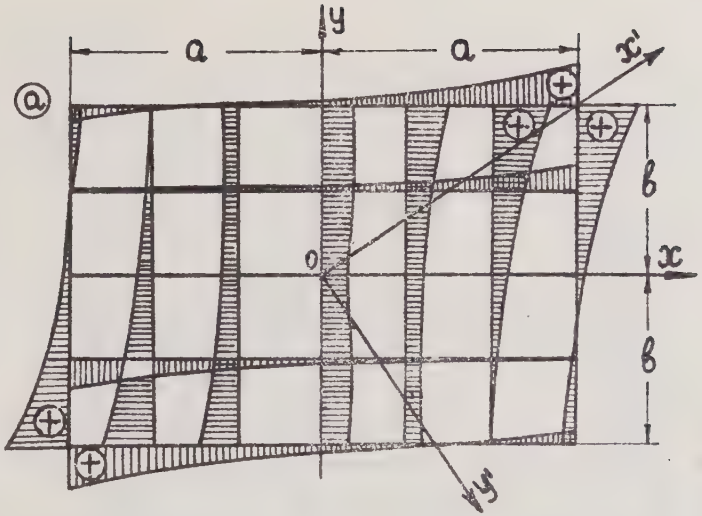


Fig.5 Diagram for the determination of the horizontal displacement



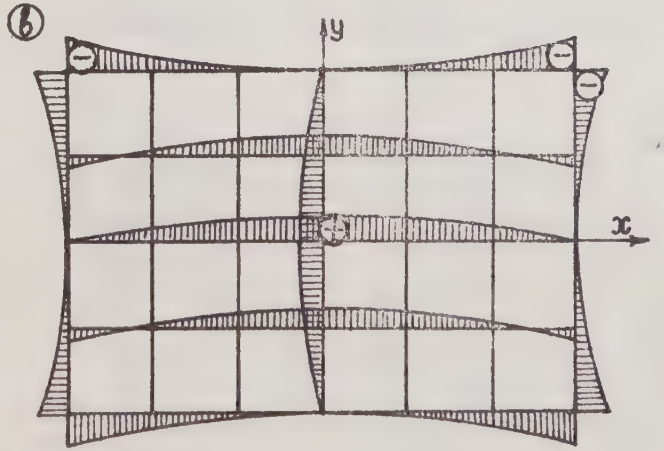


Fig.4
Normal and tangential stresses in the masonry

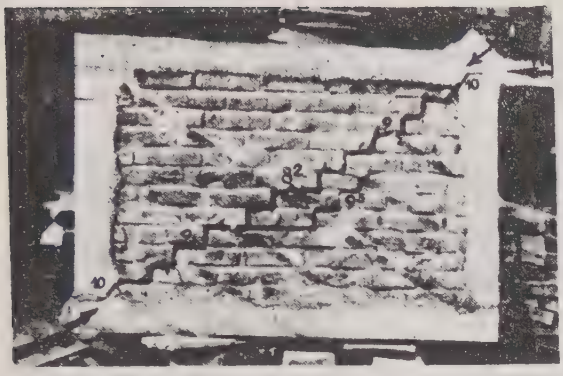


Fig.6 Diagonal fissure in one of the model discs

Fig.7 Testing a disc of a frame building; 1 - disc No.1 after the test; 2 - disc No.2 before the test

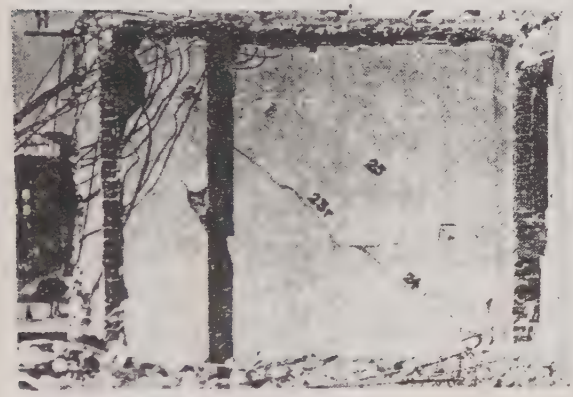


Fig.8 Diagonal fissure of a disc from a frame building

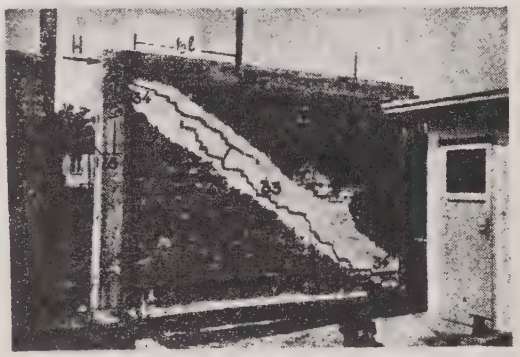


Fig.9 Diagonal fissure of a full-size disc

Fig.10 Fissures of a disc with an opening under the action of horizontal forces

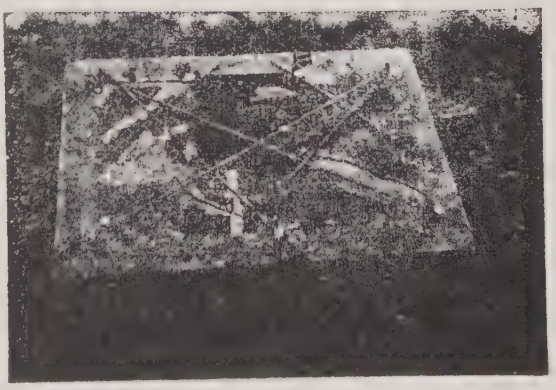


Fig.11 Fissures of a disc of lightweight monolithic concrete

Table 1

GEOMETRIC DIMENTIONS AND CHARACTERISTICS
OF THE TESTED MODEL BRICK DISCS

No	Dimensions of masonry		Opening		Cross section of the frame		Mortar		Test		Calculated capacity
	l cm	h cm	a cm	b cm	b_1 cm	h_1 cm	compression kg/cm^2	tension kg/cm^2	First fissure t.	Collapse t.	t.
1	160	108	-	-	12	12	30	2	12.2	13.5	8.2
2	-	-	160	108	12	12	-	-	1.1	1.2	-
3	160	108	-	-	12	12	20	1.47	9.5	12.2	6
4	160	108	-	-	12	12	10	1.2	7	8.2	5
5	160	108	-	-	12	12	6	0.97	4.5	6.5	4
6	-	-	160	108	12	12	-	-	1.4	1.6	-
7	160	108	-	-	12	12	20	1.4	5.5	7.5	5.8
8	160	108	-	-	12	25	25	1.6	9.5	13.3	6.6
9	-	-	-	-	12	20	-	-	1.95	2.6	-
10	190	120	70	75	12	20	28	1.7	4.30	5.3	3.8
11	190	120	35	75	12	20	25	1.6	4.80	5.2	4.14
12	190	120	95	75	12	20	25	1.6	3.60	5.1	3.2
13	190	120	50	50	12	20	25	1.5	4.70	5.7	5.3
14	190	120	35	100	12	20	20	1.2	1.35	2.8	2.2

Table 2

GEOMETRIC DIMENSIONS AND CHARACTERISTICS OF TESTED FULL-SIZE DISCS[1/]

No	Dimensions of masonry		Dimensions of opening		Cross section of the frame		Mortar		Test		Calculated capacity
	l cm	h cm	a cm	b cm	b_1 cm	h_1 cm	compression kg/cm^2	tension kg/cm^2	First fissure t.	Collapse t.	t.
1	350	280	-	-	15	60	-	0.65	23.5	2.5	15.4
2	350	280	-	-	15	60	-	0.65	21.4	23.2	15.1
3	350	280	-	-	15	60	-	0.65	7.0	8.6	7.6
4	405	265	-	-	25	15	40	1.30	39	41	38.3
5	395	255	-	-	25	25	35	1.47	31	32	30
6	385	245	-	-	25	40	38	1,50	33	33	30.6
7	395	255	-	-	25	25	15	1.12	26.6	27	22.9
8	395	255	-	-	25	25	7.3	0.71	14.8	15.2	14.5
9	296	191	-	-	20	20	35	1.47	25	26.4	22.5
10	200	127	-	-	13	13	-	1.53	10	12	7.5
11	284	255	-	-	25	25	32	1.70	23.2	24.3	26
12	174	255	-	-	25	25	30	1.65	17.2	18.8	22
13	128	255	-	-	25	25	33	1.65	14.7	15.8	21.7
14	380	240	140	150	25	40	30	1.4	14.8	15.7	13.8
15	380	240	140	150	25	40	30	1.4	12	13.2	13.8

[1/] The results are the average of three tests.

STRIKING BEHAVIOUR OF STRUCTURES IN ASSAM EARTHQUAKES.

by R.N. Joshi*

ABSTRACT.

Chapter 1: It is a prevalent practice in Assam to adopt wooden framework for structures in which timber posts are supported from isolated masonry plinth pillars. The structures in which timber posts are fixed in the masonry plinth pillars have suffered badly during earthquakes. Those in which timber posts merely rest on plinth pillars and wherein superstructure is free to move as a whole have behaved well.

Chapter 2: RECOMMENDED MODE OF CONSTRUCTION.

(i) Adopt closed frame construction with horizontal members connecting the columns at foundation level, both in soft as well as in hard strata. The construction generally stabilises the structure, as in a raft, and the horizontal members at the foundation level will prevent the relative ground motion being transferred to the structure.

(ii) For structures founded on hard strata, introduce a medium which will produce discontinuity between the foundation strata and the structure. This will put a limit on the forces and amplitude transmitted from the ground to the structure. In addition to producing effects comparable with damping, it accommodates relative movements of soil particles in the foundations.

Chapter 3: Instances are given which support the possibility of relative movement of soil particles in the foundation.

Chapter 1. REPORTED BEHAVIOUR OF STRUCTURES.

1.1. Assam is literally known as the home of earthquakes. Situated in the North East of India, it is the most unstable region in the country. It is at the foot of the Himalayas which is a young mountain still in the process of formation. All the Indian earthquakes are of tectonic origin caused by the relative movements of blocks in earth's crust along fault planes or by their sudden fracture for relieving of stresses. Assam has been the scene of a dozen of major earthquakes during the last century, including those of 1897 and 1950 which are amongst the severest earthquakes in human history.

1.2. THE COMMON TYPES OF HOUSES

(a) Semi-permenant buildings with plinth of brick masonry walls and

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superstructure consisting of split bamboo walls with plaster and a C.I. Sheet Roof.

(b) Structures consisting of wooden framework, the wooden posts going into isolated masonry pillars forming the plinth. The superstructure consists of split bamboo walls with plaster and a C.I. sheet or thatch roof. There is a clear gap between the natural ground and the ground floor. The height of plinth ranges from a foot or two to 8' or 10', the ground space in the latter case being utilised for garages and stables.

(c) Type similar to (b) above, with the only difference that the wooden posts are not fixed in the masonry plinth pillars, but merely rest on them. The superstructure is free to move as a whole with respect to the foundation.

1.3. REPORTED OBSERVATIONS.

1.3.1. In the report on Dubri earthquake of 1930, Mr. E.R. Gee (6) reports that in the (b) type of structures mentioned in 1.2 irregular oscillations were set up in the structures tending to tear away different parts of the structure from one another, resulting in heavy damage to the structures, whereas the (c) type of structures mentioned in 1.2 escaped damage. Similar observations were reported in the earthquakes of 1934 (17) and of 1950 (16).

1.3.2. Mr.Smith (14) reporting the damage caused by 1897 earthquake describes that every bit of stone work in the neighbourhood of Shillong was levelled to the ground. The only structures of the (c) type mentioned in 1.2 were stables and outhouses and these escaped untouched. Mr.R.D.Oldham (14) who is to be credited with the pioneer study of Indian earthquakes describes that while Government buildings fared badly it was curious to note that the houses of the American Baptist Mission (being of the 'C' type mentioned in 1.2) withstood the shocks with complete immunity. The movements of the framed superstructure relative to some portions in the plinth were measured by the marks on the underside of the frame to be 5".

1.4. DAMAGE TO STRUCTURES FOUNDED ON ROCK.

1.4.1. It has been observed (1) that there was apparently no bearing of damage to structures to the strata on which they were founded and a number of houses whose foundations were on solid rock were damaged while others on alluvium were not damaged.

1.4.2. Milne (13) has cited instances of the New Zealand earthquake of 1931 and the Italian earthquake of 1930 wherein greater damage to structures founded on rock than otherwise was definitely noticed. Mr.J.R. Freeman (5) has also drawn pointed attention to this paradox. Statistical study of earthquake damage in Japan (12) also indicates that rigid frame structures suffer greater damage if founded on rock than if founded on soft strata.

1.5. EARTHQUAKE SHADOWS.

1.5.1. In the report of the Shrimangal earthquake of 1920 (19) attention has been drawn to the extra-ordinary escape of Bharanara and other Tea

Estates owing to their being sheltered by hills. The experience is by no means unusual. Charles Davison (3) has mentioned that where alluvium strata abuts against a hillock, the earthquake wave energy is dissipated at the junction with the result that earthquake damage is precipitated at the foot of hills. A similar observation was made by the author (10) after survey of the damage to the city of Anjar in the Kutch earthquake of 1956, where a part of the town at the foot of a hillock was found to be susceptible to earthquake damage on more than one occassion.

1.5.2. Mr. C.S. Fox (4) suggests that a deep wide trench round an area would shelter the same from earthquake damage. An artificial barrier to precipitate damage with a view to sheltering specific areas from damage looks feasible.

Chapter 2: RECOMMENDED MODE OF CONSTRUCTION.

2.1. STRUCTURES TO BE FOUNDED ON SOFT STRATA.

2.1.1. Based upon the observations regarding behaviour of certain types of structures during Assam Earthquakes, it is proposed to make here certain recommendations regarding the mode of construction to be adopted in general in all earthquake affected areas.

2.1.2. It is recommended that the frame work construction be adopted in preference to structures with load bearing walls.

2.1.3. It is recommended that the framework should comprise of a closed frame, i.e. there should be closing horizontal connecting members at the bottom. Such framework as recommended is depicted in figure 1.a as against framework standing on unconnected foundation legs as shown in figure 1.b. of the customary type. Raft foundation can be said to be a more elaborate form of closed framework construction, the former being represented in the latter as a skeleton.

2.1.4. Even if raft foundation is taken, it is not advisable to form a basement by constructing end walls between the raft and the ground. It would be preferable just to close the gap by an external curtain wall hanging from the plinth beam upto somewhat below ground level; otherwise a clear gap between the plinth and the ground may be left supporting the superstructure on columns as in the case of the Shillong type of construction.

2.2. STRUCTURES TO BE FOUNDED ON ROCK OR ON HARD STRATA.

2.2.1. It is recommended that they should be of framework type and that the foundations of columns should <u>not</u> be thoroughly keyed into the rock, i.e. the columns should <u>not</u> be "fixed" or "encastered" in the foundations. Columns should merely rest on the hard strata in a notch provided in the latter. Some plastic material can close the gaps on the notch which would permit slight movement of the foundation strata with respect to the structure. It would be enough if a latitude of 1/2" to 1" is provided all round the notch for such a movement.

2.2.2. It would be desirable to introduce a layer of accommodating material between the surface of the rock and the bottom of the column. Such insulating layer may be formed of asphaltic concrete, neoprene rubber sheeting or some other suitable material, the purpose of such insulation being to break the continuity of column with the foundation strata and to permit a play between them to a small extent.

2.2.3. It is further suggested that a closed framework be formed even in the case of foundations on rock or on hard strata, as in the case of soft strata (2.1.3.) with the only difference that the closing horizontal members shall not rest on rock, but shall be slightly raised so as to reduce resistance to the movement of the foundation strata with respect to the structure. The construction is indicated in figure 2.

2.2.4. In case one does not like to permit the structure to merely rest on hard strata and desires to provide a positive anchorage in the rock (or in the hard strata), it is recommended to adopt columns 'hinged' at the base as shown in figure 3.

2.2.5. Still another method would be to introduce the accommodating material between the top of R.C.C. column and the plinth beam. The structure would then behave very similar to Shillong type of construction by absorbing the relative motion in the insulating material at plinth level.

2.3. CENTRAL IDEA IN THE RECOMMENDATIONS.

2.3.1. The central idea is to break the continuity or rigidity of the structure with the foundation strata on which it is supported. The discontinuity will in the first instance considerably reduce movements and shocks transferred to the structure, the insulating material acting as a buffer. In addition it will also permit differential movement, if any, between the soil under different portions of the structure.

2.3.2. The closing members at the bottom act as braces at lower levels and interconnect the foundations. Any forces that are produced by the differential ground movement during the shock are overcome as direct forces of compression and tension in the closing members, and are prevented from creating undesirable stresses due to heavy bending moments and shears. As a result, motion transmitted to the structure shall be a sliding or a rocking movement of the structure as a whole and there will be no tendency for different parts of the structure to tear or shear away from one another.

2.4. STATIC EFFECTS OF EARTHQUAKES ON STRUCTURES.

2.4.1. It would be instructive to study the static and dynamic effects of earthquakes on structures . Such a study will reveal the difference between the expected behaviour of the recommended types of structures and the customary ones.

2.4.2. The forces on account of earthquakes which need consideration are as under:

(a) Equivalent static effect, (b) Dynamic effects and (c) effects due to differential ground movement.

2.4.3. The effect of sudden movement of the ground on the structure is equivalent in magnitude to the effect of the inertia force on the structure produced by the acceleration with which the ground is moving at any instant. These forces are commonly taken into consideration in all designs; certain aspects of such forces are dealt with presently.

2.4.4. When the ground on which a structure stands starts moving, say with an acceleration α, this acceleration is transmitted to the structure through and on account of the continuity of the structure with the ground. Having no continuity with the ground, an aeroplane flying in the air will not suffer any acceleration due to ground shocks. As long as the continuity between the foundation and the structure is maintained, the lateral force operative on the structure of mass M is $M\alpha$, and the acceleration produced in the structure (α') is the same as α. If the bond resistance or the frictional resistance between the ground and the structure (F) is less than $M\alpha$, sliding between the two takes place, and in that case the lateral force operative on the structure does not reach the full value of $M\alpha$, but is limited by the value of F. The acceleration of the structure α' that would be produced when sliding between the ground and the structure takes place is the acceleration which a force F would produce on a mass M. F, the resisting force of sliding friction is equal to μMg where μ is the coefficient of friction; the acceleration in the structure would then be

$$\alpha' = \mu \cdot g \qquad \ldots\ (1)$$

For calculating the actual acceleration and forces on different parts of the structure the appropriate reductions for damping and amplifications for increased heights etc., will have to be considered in addition.

2.4.5. The author in a paper written jointly with Mr. S.B.Joshi (11) has shown that friction between two surfaces comprises of resistance to work against gravity in riding over irregularities, and, or resistance in shearing off the irregularities. When structure is founded on soft material, the frictional resistance is replaced by the shearing within the soft material and is equal to S.A, where S is the shearing resistance per unit of Area and A is the area of shearing. Thus in such a case the force transmitted to the structure $F = SA = M\alpha'$. The effective acceleration produced in the structure will then be $\frac{SA}{M}$

$$\alpha' = SA/M \qquad \ldots\ (2)$$

2.4.6. The expressions for acceleration derived in 2.4.4. and 2.4.5 are significant.

α' the acceleration of structure = acceleration of the ground α and is limited by $\alpha' = \mu \cdot g$ .... (1)

or $\alpha' = \frac{S\,A}{M}$ .... (2)

Acceleration in equation 1 is independent of Mass, while that in

equation 2 is inversely proportional to the Mass of the structure. It can, therefore, be concluded that as far as resulting forces and accelerations on the structure are concerned, increased Mass or inertia of a structure is definitely helpful in the case of foundations in soft strata, i.e. where equation 2 applies.

Of course the increase of mass should not be such as to add to the debris in case of failure, for very often the damage to life and property arises more out of the debris than anything else.

2.4.7. Prof. Suyehiro (20) has shown that 'by reason of its inertia, the movement of a rigid building on a massive foundation may be much smaller than that of sand or mud on which the foundation rests. Soft yielding sand or mud sways to and fro beneath the foundation without imparting to it vibrations of large magnitudes'. The view confirms observations in 2.4.6. When the maximum force transmitted to the structure is determined by S.A, as in equation 2, the fill on the raft artificially increases the effective mass of the structure as a whole and hence reducesthe acceleration of the structure. The fill on the raft, however, does not increase the effective force producing stresses in the framework of the structure, but on the other hand decreases the effective force by reducing the impressed acceleration.

Further the raft distributes the load on a large area, makes the structure bottom heavy, adds greatly to its general stability and permits rocking and sliding of the structure as a whole. All these benefits make a raft a very effective construction against earthquake damage. A closed frame structure inherits the above benefits, some partially and others to the full extent.

2.4.8. At the same time that the raft increases the effective mass, it also increases the base area A. Thus according to equation 2, increase of A increases α', while increase of M reduces α'. Perhaps a closed frame structure as recommended herein strikes a good mean. Actually it has an advantage over a raft in that the former has less base area and transfers less lateral force to the structure than in the case of a raft. A raft without a fill over it, as in the case of a basement, would, however, be a positive disadvantage, as it would increase the lateral force transmitted to the structure (S.A), without increasing the mass and hence without reducing the acceleration on the structure. In addition the side walls of the basement expose large area of obstruction to earthquake waves. This substantiates 2.1.4.

2.4.9. As shown earlier, the basic maximum acceleration produced on a structure during an earthquake is less than the maximum ground acceleration. The actual motion of the ground, it is known, is complex. The actual motion of the base of the structure is still more complex in view of the fact that the instantaneous acceleration of the structure is not only dependent on the instantaneous ground acceleration but also on the relative velocity between the two. It is to be expected that with the reduced maximum acceleration of the base of the structure, its maximum amplitude and its average velocity will be less than the corresponding

values for the ground motion. The effect of the reduced maximum amplitude of the base of the structure is dealt with in 2.5.2.

2.4.10. If V is the average velocity of soil particle during a dominant wave in an earthquake, the energy input of the structure is $MV^2/2$. The energy is dissipated in several ways such as damping and in causing the elastic and plastic strains in the material of the structure. If the energy input is beyond what the structure can bear, some dissipation of energy takes place in cracking and in causing other damage. Now if during the transmission of motion from the ground to the structure, sliding between the ground and the structure (2.4.4.) or shearing in the foundation material (2.4.5.) takes place, the average velocity transferred to the structure is less than the average velocity of the ground (2.4.9.). The energy transferred to structures provided with discontinuity as recommended is therefore less and consequently less in the chance of damage to such structures.

2.5. DYNAMIC EFFECT OF VIBRATIONS.

2.5.1. The natural period of vibration of a structure considered as a shear column depends upon the end condition at the base, and will be much larger if the base is hinged rather than fixed in the ground. Study of earthquake spectra shows that response base shear of a building decreases almost inversely with its natural period. Moreover higher natural period reduces the possibility of synchronization of dominant period waves.

2.5.2. The deflections from vertical position, bending moment and shear produced at different altitudes in a structure are all direct functions of the maximum amplitude of vibration at the base of structure (2). The discontinuity between the ground and the structure reduces the amplitude of vibration during its transmission from the ground to the base of the structure. The discontinuity in recommended construction therefore results in reducing the deflections from the vertical, bending moment and shear at different altitudes of a structure during strong ground motion.

2.6. EFFECTS DUE TO DIFFERENTIAL GROUND MOVEMENT.

2.6.1. In addition to the effects on the structure as a whole, it is necessary to consider the effect of differential movement below two neighbourning columns of a structure. The question about the possibility of such differential movement is discussed in Chapter 3.

2.6.2. With a view to simplification forces on a portal frame are considered here. The inferences will be applicable to rigid framed structures in general. Three cases will be considered, namely, when the frame is (a) fixed, (b) hinged or (c) sliding on the ground. It is assumed that the ground at the supports is moving and that there is a relative horizontal displacement Da, Db and Dc respectively in the plane of the frame. It is furtheassumed that the moment of inertia and length of all the members of the portal frame is the same.

(a) Portal frame fixed in the ground (figure 4a)

$$\text{Bending Moment} = \frac{2EI}{L^2} \cdot Da \qquad \ldots (3)$$

$$\text{And shear} = H = \frac{3EI}{L^3} \cdot Da \qquad \ldots (4)$$

(b) Portal frame hinged to the ground (figure 4b)

$$\text{Bending Moment} = \frac{0.6\ EI}{L^2} \cdot Db \qquad \ldots (5)$$

$$\text{And shear} = H = \frac{0.6\ EI}{L^3} \cdot Db \qquad \ldots (6)$$

(c) Portal frame with aliding supports, the resistance to sliding being F when sliding begins (figure 4 c).

$$\text{Shear} = F \qquad \ldots (7)$$

$$Dc = \frac{5}{3} \cdot \frac{FL^3}{EI}$$

$$\text{Bending Moment} = FL = \frac{0.6\ EI}{L^2} \cdot Dc \qquad \ldots (8)$$

Values of Dc and F are not more than the values of Db and H, the corresponding values being equal till the sliding in (c) takes place.

Bending Moments and shears in the three cases depend upon the moment of inertia and length of members; but the results obtained above are indicative of the influence of base conditions on the same.

It will thus be seen from equations 3 to 8 that columns with hinged bases will produce less severe stresses than columns with fixed bases. However, columns with sliding bases will produce still less stresses, their maximum being equal to the case when the columns are hinged.

The equations also explain the paradox that R.C.C. riged framed structures are safer when founded on soft strata than when founded on rock, for in the former case equations 1,2,7 and 8 operate whereas in the latter case equations 3 and 4 operate.

2.7. CORROBORATION WITH RESULTS.

2.7.1. In the foundations of some tall structures it is a practice in the United States of America to introduce a 3 feet layer of gravel between foundation rock and the structure (2). The purpose of such layer is to

break the continuity between hard foundation strata and the structure. The practice confirms the principle of the recommendations stated in 2.3. The hinged condition at the base also helps to increase the natural period of frequency of the structure.

2.7.2. Mr.Jacob J.Creskoff (2) has recommended the adoption of interconnected foundations, where raft foundation is prohibitive in cost. This is in confirmity with the recommendations made herein.

2.7.3. Behaviour of the Shillong type of structuresin Assam have well exemplified the immunity of such type of construction, i.e. of structures which were not fixed to the ground but were free to move as a whole. The recommendations made in this paper are merely the extension of the principle.

2.7.4. Raft construction is almost universally recommended and has yielded splendid results. As mentioned in 2.3.1. raft is merely an elaborate form of a closed framework type of construction; whatever applies to a raft applies to a closed frame structure.

2.7.5. To illustrate the point, it may be well to give some parallels from daily observations. A table or a stool with legs connected near the base behave well in dragging and shaking. So also does a box.

The heavy forces of nature are best tackled by curbing them than by total obstruction. Tiny weeds can stand a flood better than a stout tree.

Chapter 3. DIFFERENTIAL GROUND MOTION

3.1. SURFACE WAVES.

3.1.1. Differential ground motion within the area occupied by a structure due to passage of Primary and Secondary waves would be too small to affedt the structure, because of the high magnitude of wavelengths of such waves. From the point of view of differential ground movements, surface waves are important.

3.1.2. Though surface waves were first predicted theoretically by Poisson as early as 1829, and more elaborately explained by Rayleigh in 1887, it was not till 1900 that various types of surface waves were analysed by actual records due to the distinguished work of R.D. Oldham on Assam Earthquakes. It was observed in Assam Earthquakes that the rice crop was falling and rising, as the waves passed, their speed though being decidedly faster than what a man could walk was not as fast as he could run. Mr.Oldham estimates the wavelength to be 30 feet and the height of waves as one foot (14). During the 1811 earthquake in the Central Mississipi area, the ground was observed to undulate in waves several feet high and the depressions between successive waves were clearly visible (9). During the Charlestone earthquake of 1886 in South California, waves about 2 feet high were observed moving rapidly and shadows cast by successive wave crests could be seen distinctly in the gas light. 'One of the most

alarming and disastrous features of a great earthquake near its place of origin', observes Arthur Holmes (8), 'is the passage of large surface waves over the ground which is thrown into overchanging undulations'. F.Kingdon Ward, a western traveller, who was caught in the Assam earthquake of 1950 gives a very vivid description of the movement he experienced saying that the solid mountains were in the grip of a force that was shaking them as a terrier shakes a rat. (21).

3.2. DISPLACEMENT OF OBJECTS.

3.2.1. Displacement of objects during earthquakes leaves no doubt of the differential ground motion involved. A few illustrations are given. It might be noted that some of the observations are at fairly distant places from the epicentre being in isoseists nos.3 and 4, indicating that differential ground movements takes place over a vast area.

3.2.2. An extremely interesting case has been reported by Murray Stewart (19) while reporting on the Shrimangal earthquake of 1920. At Kishoreganj in isoseist no.3 was a stack of loose bricks about four feet wide and five feet high, 120 feet in length in the North South direction. The bricks had fallen in places along the western face of the stack, not all the way along the face, but at definite intervals, there being a space of 31 ft. unfallen between the two consecutive points where bricks had fallen. A possible explanation for equal intervals of 31 ft. between spots of disturbance is the formation of nodes and antinodes due to the combination of two simultaneous waves arising from the sympathetic shocks off Madras and Arakan reported during the said earthquake which having components in the same direction must have caused maximum displacements at points where the two waves strengthen each other. Alternatively formation of antinodes may be due to some local reflection and refraction. In the Charlestone earthquake of 1886 in South California (9), surface ground waves were reported to be crossing others diagonally and giving the effect of a chopping sea. The action on the stack of bricks appears similar. Whatever the explanation, the fact of loose bricks not falling all along but only at certain spots indicates that movement at different places though near to each other are different.

3.2.3. At Gourigram in Isoseist no.4 of the Shrimangal earthquake (19), two sides of a triangle of a railway line used as a turntable were considerably bent, one rail being found to have a curvature equivalent to a radius of 30 ft. The base of the triangle was at right angles to the the direction of the wave and was not affected. Bending of rails has in fact been very frequently observed.

3.2.4. Reporting on the Cachar earthquake of 1869 (15), Mr.Oldham describes that a Hindu temple under the shelter of a banyan tree was quite uninjured in the neighbourhood of a well-built jail wall which lay in ruins. Oldham explains the miraculous escape being due to the binding effects of the wide spread roots of the tree which tied the whole ground together. This instance indicates the protection from damage by preventing differential ground motion.

3.2.5. Oldham reports of relative displacement of two parallel tombs at Cherrapunji in the earthquake of 1897. The visible signs on the ground indicated that both the tombs had shifted towards each other, one by 18" and the other by 10".

3.3. ROTATION OF OBJECTS.

3.3.1. Several observations of rotation of objects have been reported in Assam Earthquakes. A few notable instances are given below:

3.3.2. A billiard table at Lalchand Tea Estate rotated through a few inches in the clockwise direction (19). At Luskerpore Tea Estate vertical flat irons of the frames of the leaf house were twisted. In isoseist no.3 Langla Marager's bunglow a 400 day clock which was facing East and standing on the top of a whatnot rotated in a clockwise direction and came to rest facing North. An iron safe facing West was jerked out towards the west and was left facing North West.

An obervation of special interest reported by Oldham is the rotation of all the eight columns of an octogonal thermometer shed at Dubri, everyone rotated round its own vertical axis in the clockwise direction through an angle varying from 5 degrees to 12 degrees.

3.3.3. Pillars and monuments being supported on a single base, without any branches or overhanging projections are simple structures which would involve the least possible complication of stress conditions. Acting almost like a pendulum they keep a faithful register of forces coming into operation during earthquakes. Where instrumental observations are lacking they can be a valuable source of information.

3.3.4. Some very interesting observations have been reported by Oldham.

In a monument to George Inglis at Chhatak rising to a height of 60 ft. from a base of 12 ft. square, the topmost 15 ft. of the pillar was thrown off and the block of the next 22 ft. height had twisted through 30 degrees in the anti-clockwise direction. The section of the pillar at the plane of rotation was 9 ft. square. During rotation, the twisted portion had shifted 1'-2" towards N 55° W.

At Cherrapunji out of two pillars standing close to each other, one twisted anti-clockwise through 5 degrees while the other twisted clockwise through 4 degrees. At Shillong, a gate pillar of Fernadale Hotel turned anti-clockwise on its base through 4 degrees and again above the lowermost stone through another 2 degrees. A tomb pillar at Gauhati was broken higher up and had twisted clockwise through 48 degrees. Mr. Gill recorded during Dubri earthquake twisting of the topmost three feet of a fifteen feet high pillar through 20 degrees.

Opposite twisting of neighbouring pillars has been reported in some cases and is likely to be thought as baffling. Such opposite rotation was also reported of two obelisks standing side by side in the Calabria earthquake of 1783.

3.3.5. Millet, Gray and Oldham tried to explain rotation of pillars, each in his own way; but the common tune appears to be that changing of directions in rapid succession results in lifting of the centre of gravity

slightly on one edge and being pulled in another direction in the meanwhile. This suggests a sort of vertex motion or some complex combination of S.H.M. for individual soil particles. The possibility of such complex combination of simple harmonic motion of individual particles can be conjectured by superimposition of N-S, E-W and V-H seismograms of a point vibrating under a strong ground motion.

3.3.6. A vortex motion or a complex combination of S.H.M. of individual soil particle explains the rotation of bodies, twisting of monuments as also the twisting of all the eight columns of the thermometer shed (3.3.2). It is further to be appreciated that the simultaneous rotation or twisting of two points each about its own vertical axis implies a combination of rotation about a common axis and a differential motion of translation between the two points.

3.4. SEISMOGRAPHIC DATA.

3.4.1. The seismographs of different stations in the town of Yakohama have not produced identical seismograms with a short time lag for the same earthquake (12). This proves that actual ground motion at two places within a short distance of each other differs considerably, i.e. so to say the differential movement between them does exist.

3.4.2. As a matter of fact, installation of seismographic instruments has to be done well below the ground surface, so that erratic movements at the surface are eliminated. Thus at very shallow depths of say 4' to 8' where foundations of several structures may be founded, differential soil movement is more likely to prevail than not.

3.5. GROUND UPHEAVELS DURING EARTHQUAKES.

3.5.1. Ground upheavels and depressions over a considerable expanse of land and water during earthquakes have been observed. After a lot of survey operation after the Assam earthquake of 1897, Mr. Bond found that heights of stations had risen by amounts varying from 6' to 24' and in one case there was a subsidance of 4'. A horizontal displacement of several feet of vast area was brought about during the earthquake. The greatest throw of the chedrang fault reported in Assam was 35 ft, the greatest ground uplift ever recorded was during the Alaska earthquake of 1899 where the coast was elevated by 47'-4" at one point. One of the largest submarine movements on a slope West of Greece in 1893 raised the sea bed from 1600 to 2300 ft.

3.5.2. With such heavy and expansive ground movements during earthquakes, relative movements cannot be ruled out of possibility.

3.6. GENERAL.

3.6.1. There is little doubt that the nature of the strata immediately below the surface greatly modifies the amplitude and speed of surface undulations (4). The seismograms at three stations within 1 Kilometer distance of each other in Tokyo (12) confirm the above statement. Because of the complex nature of earthquake waves and the heterogeneous character

of the earth, the movement of a particle during an earthquake is extremely complex (18). A differential movement of soil particles within short distances is therefore to be expected.

3.6.2. The above illustrations and discussion strongly suggest the possibility of differential ground motion in foundations. It is therefore necessary to take them into consideration in designing earthquake resistant structures and also to take steps to counteract the same.

It is submitted that the recommended mode of construction takes full notice of such differential ground movements.

CONCLUSION

Reference is invited to the abstract

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NOMENCLATURE.

A Area of shearing.

Da Horizontal deflection of the portal frame, when fixed (fig.4a)

Db Horizontal deflection of the portal frame, when hinged (fig.4b)

Dc Horizontal deflection of the portal frame when supported on roller (figure 4c)

E Modulous of Elasticity.

F Force of resistance to sliding.

g Acceleration due to gravity.

H Horizontal pull on the frame when fixed or hinged.

I Moment of inertia.

L Length of the vertical member of the portal frame.

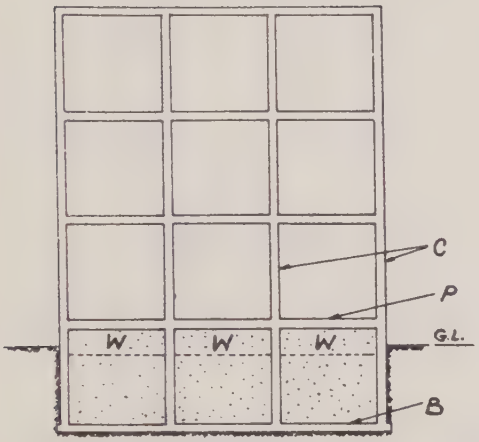
M Mass of the structure.

S Shearing strength per unit area.

V Average velocity of soil particle.

α Max.Acceleration of ground during an earthquake.

α' Max.Acceleration of the structure during an earthquake.



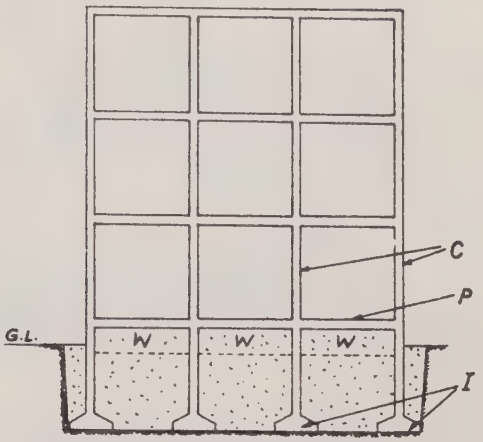
B: BOTTOM CLOSING MEMBER OF FRAMEWORK.

C: COLUMNS.

P: PLINTH BEAM.

W: CURTAIN WALL HANGING FROM PLINTH BEAM.

FIG. 1a (REF. 2.1.3.)



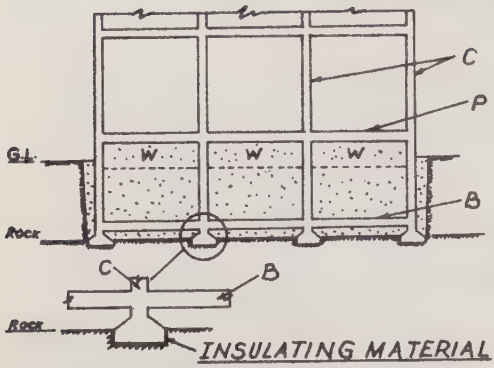
C: COLUMNS

I: ISOLATED FOUNDATIONS OF COLUMNS

P: PLINTH BEAM

W CURTAIN WALL HANGING FROM PLINTH BEAM

FIG. 1b (REF. 2.1.5)



B: BOTTOM CLOSING MEMBER OF FRAMEWORK

C: COLUMS

P: PLINTH BEAM

W: CURTAIN WALL HANGING FROM PLINTH BEAM

FIG. 2 (REF. 2.2.3)

FIG. 3 (REF. 2.2.4)

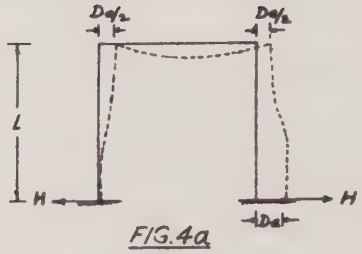


FIG. 4a

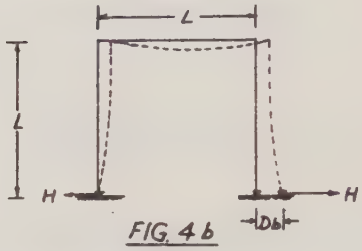


FIG. 4b

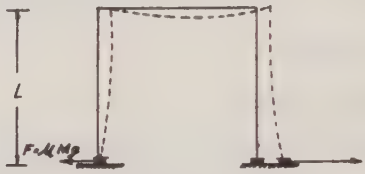


FIG. 4c (REF. 2.6.2)

ENGINEERING ASPECTS OF EARTHQUAKES IN RUMANIA IN THE LIGHT OF MODERN INVESTIGATIONS

by

Aurel A.Beleş[x] and Mihail Ifrim[xx]

I, IMPORTANCE AND ROLE OF ENGINEERING SEISMOLOGY

Among the natural calamities striking mankind earthquakes are the most dreadful because of the huge damages they provoke. In a few seconds, important losses of human lives and enormous material damages are recorded.

History mentions numerous earthquakes which deeply impressed human remembrance.

For instance the earthquake in Lisbon (nov.1,1755) caused over 3o,ooo dead and a great destruction of the town. The earthquake in Tokio and Yokohama is reported to have caused the death of more than 1oo,ooo people and destroyed more than 125,ooo buildings through quake shock and 45o,ooo by fire. The Assam quake (Jan.12,1897) is considered until now as having shown the greatest intensity and area of destruction ever recorded in the world, whilst the San Francisco quake of 19o6 showed, for the first time, the disastrous effect of earthquake shocks on the modern building systems and the importance of fire as a consequence of quakes on big and crowded towns.

Considering the great losses of life and damages caused by quakes it is easy to see why mankind tried to find means to suppress or at least to reduce the disastrous effects of earthquakes.

We must stress that the great number of dead is due mainly to the failure of buildings.

Therefore the problem of buildings resisting to earthquakes is of a particular importance and actuality, so much the more, as the overcrowding of towns increases the danger for the inhabitants.

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The first protection measures against quake damages were based on elementary observations and had no scientific support.

They were restricted to different requires referring usually to the building material employed and to the type of constructions

At a certain moment, wooden buildings were in great favour, on account of their resistance to shocks and to their light weight. But soon they proved dangerous on account of fire which always follows earthquakes.

The first work concerning designing of quake resistant buildings are due to the French engineer Lescasse, who published in 1877 in Paris a book, "Etude sur les constructions japonaises et sur les constructions en général au point de vue des tremblements de terre et description d'un système destiné à donner une grande sécurité aux constructions en maçonnerie". This study was based on the effects of earthquakes in Japan.

Later on, John Milne published in Transactions of the Seismological Society of Japan, Tokyo, 1880, a paper: "The earthquake in Japan of February 22,1880" and two years afterwards "Construction in earthquake countries (ibidem, vol.XIV).

Of a wide practical importance were the studies of F.Omori published at the end of the XIX century in Japan. These studies have opened the way to scientific preoccupations in the field of antiseismic structures. The elaboration of Omori's "static method" means an important technical improvement in this direction.

The greatest difficulty the investigators met with, was the appreciation of the mechanical elements characterizing the seismic motion, which were strictly necessary to pass to the designing of constructions.

Along with the progress of science and the development of new designing methods of structures on one side, and the using of steel and reinforced concrete for structures frames on the other side, the problem of earthquake resisting structures could receive a scientific basis.

The investigations carried out in Japan (F.Omori) have known a great soaring after the earthquake in San Francisco (1906) when the modern structure system on resistant frames has shown its qualities in a strong earthquake.

Later on, when the record of the characteristics of seismic wave by means of modern instruments was made possible, the premisses of investigations closer to the real

phenomenon were created. Nevertheless, we shall mention that true engineering seismology is of recent date, as a consequence of the sustained efforts of a great number of engineers and scientists in the world.

The first world conference on engineering seismology, which discussed the problem of the destroying action of earthquakes on structures took place in San Francisco in 1956. In the same year, In Moscow, were presented the works of a conference with similar character.

The impulse given by these important scientific manifestations determined all earthquake visited countries to pay to the engineering seismology greater attention, using the most recent discoveries and investigation possibilities offered by science and its application.

Owing to the organization of the IInd world Conference on Engineering Seismology in Japan, the country with valuable contributions for the creation of a new science on engineering seismology, it seems that the exchange of opinions and informations on international level receives a permanent character. This fact is gratifying to those who work at the delicate mission to diminish or even to suppress the disastrous effects of earthquakes.

II. ENGINEERING ANALYSIS OF THE EARTHOUAKE IN RUMANIA OF NOVEMBER 10, 1940

Rumania, generally, has not been considered as an intense seismic country, although during centuries a series of earthquakes have been mentioned as having produced important damages. Nevertheless, it cannot be spoken of catastrophes like those in Portugal,Japan,America,India,a.s.o.

We can mention for its intensity, the earthquake in 1445 which has committed ravages and the shock of which was felt strongly enough even in Moscow.

The earthquake in 1472 among other damages distroyed the Church of the Neamţu Monastry, renowned art monument of our country and the earthquake in 1683, which caused the collapsing of the Suceava fortress towers, the ancient chief city of Moldavia.

We also mention the earthquake which in 1738 damaged the lordly palast of Bucharest, and the earthquake in 1802, which caused the collapse of the "Colţea Tower" a remarkable building in Bucharest at that time.

Due to the fact that in Bucharest and in other towns of the country, there were no important constructions, the damages produced even by the strong earthquakes caused no important destructions.

More strongly have been felt the effects of the earthquakes in our century, which is characterized by an important soaring in constructions.

Important earthquakes recorded in this century are those in 1912, 1916, 1929 and 1938, but without great evidence of distress.

The earthquake which deeply impressed the inhabitants of Rumania was that of November 1o, 194o. Because of this earthquake Rumania has been considered since among the countries with pronounced seismicity.

The damages produced on constructions situated in various regions of the country and the extensive region subjected to earthquake arose among the engineers a well-sustained activity for analysing the causes of damages and destructions and for the design of antiseismic structures.

Several earthquakes centers have been identified till now, but the most important which has produced also the earthquake of November 1o,194o is situated in the Vrancea Mountains. This center is defined by the geographic coordinata 45.9°, northern latitude and 26.5° eastern longitude, being situated at the depth of nearly 15o km (fig.1). Because of this depth, its effects are felt on a large zone.

Measurements regarding the characteristics of the seismic waves were not performed because the instruments of the Institute of Seismology in Bucharest were put out of order at the first shocks of the earthquake.

The intensity of this earthquake was stated by means of the seismic scale Mercalli-Cancani-Sieberg.

The medium degree of earthquake was 8 and it affected nearly 8o,ooo km^2. In Bucharest it was estimated at 8-9 degree, and within the epicentral region and other isolated points, at degree 1o. According to Gutenberg-Richter scale, this earthquake had the magnitude 7, being of class B. The Carpathian Chain protected, in a certain measure, the Transylvanian Plateau, and the northern, north-western and south-western regions of the country (fig.2).

In order to state the earthquake intensity and to establish the isoseistics, nearly 5ooo reports from various parts of the country were used, being corroborated afterwards with other personal investigations performed by seismologic researchers.

We mention that in the epicentral region some villages were totally destroyed and in Panciu and Focşani, towns in the neighbourhood of the epicenter, extremely great da-

mages were noticed. All the destroyed or damaged buildings showed a very weak execution and a defective engineering conception.

The most heavy accidents of the earthquake of Nov.1o, 194o took place in Bucharest. Here one of the tallest civil buildings (the Carlton building)of very recent construction date, having two undergrounds and 11 stories on a reinforced concrete structure collapsed completely burying under its ruins almost all the lodgers of the building, i.d. over 1oo persons. With regard to the amplitude of this disaster, it seems that in the history of the concrete constructions, this accident represents one of the greatest destructions.

The great number of dead was also due to the fact that the accident took place in the night, at 4 o'clock in the morning, local hour.

In Bucharest other buildings of reinforced concrete were also damaged, as well as smaller brick-buildings. We must point out that reinforced concrete as resistant material for tall constructions was introduced in Bucharest only after 191o. In general, these structures were calculated only for vertical loads and very few were verified for wind action. Generally they were stiffened with brick walls, which offered a lateral rigidity sufficient for wind stresses.

No assurance whatever of these constructions against earthquake had been taken into account, the intense earthquakes having occurred tenths of years before, when no constructions of such height or importance existed in the town.

The most important discussions which followed the earthquake of Nov.1o,194o were caused by the collapse of the Carlton building. The real causes of the collapse could not be established on account of the complete destruction of the reinforced concrete resistant frame followed by a big fire, but some assumptions could be made as is shown later.

It is interesting to note that opposite the Carlton building was another building, 6 stories high, having only concrete floors and load-bearing brick masonry. This building resisted in very good conditions, although the soil conditions were the same and the absence of a resistant skeleton rendered it more sensible to earthquake shocks.Other buildings of the neighbourhood too, as tall as the Carlton building suffered only very slight damages.

An examination from the engineering side of all the various damages and especially of the causes that produced the failure of the Carlton building, was performed by

Prof.Eng.Aurel A.Beleş,member of the R.P.R.Academy, In the two papers published immediately after the quake /1/, /2/ are exposed in a detailed manner the causes of the damages and the destruction of different structures in Bucharest.

In connection with the complete collapse of the Carlton building, the essential cause was the conception on which the design of the reinforced concrete frame had been based. As there were shops at the ground-floor, in order to get as big shop-windows as possible, some of the columns were reduced to transversal reinforced concrete walls only 22 cm in width, but 2oo m in length.

After clearing away the ruins of the building in the cellar of the building, one of these concrete columns was found intact. It seems that by the earthquake shock, the column was cut at its base and on account of insufficient connection of the reinforcement the column was removed on the reinforced plate, perforated it and fell in the cellar, drawing the whole building after it.

The Column having the heating element still attached can be seen in the figure (fig. 3 - 7).

Consequently, the failure of the Carlton building may be attributed to the shear caused by the seismic shock which produced the section of columns and afterwards the complete failure of the building. To the complete destruction also contributed the imperfect mode of transmitting the loads to the foundation due to the lack of continuity of all the columns.

Generally, for structures resting on a concrete skeleton, it was stated a series of faults due to some execution defects. For instance, there were found columns whose reinforcement bars were bound in unfitted places (fig.8, 9) other columns with too thin and too distanced ties (fig.10, 11), some with non-corresponding structural joints , and insufficient reinforcement or with a diameter (fig.12, 13) too small - at same columns - relative to the importance of forces they had to support. On that account, many local yields have been recorded, which caused only local ruptures without collapse.

In case of structures consisting of load-bearing masonry, important faults were recorded in those constructions where the mortar was insufficiently hardened or where the vertical brick joints were not filled. These were the causes that a series of modern buildings although tied with concrete floors showed strong degradations of the masonry (fig. 14, 15). The presence of wooden floors producing by the supports of the beams a discontinuity in the masonry,

caused the rupture of walls, In general, the apparition of the classical X-shaped ruptures as a consequence of strong shear forces was noticed (fig. 16 - 19).

We mention the fact that structures with a concrete skeleton suffered more as compared to structures with load-bearing masonry and concrete floors. A satisfactory behaviour showed the common old buildings of the last century when concrete was not yet introduced in Rumania, but which had sufficient ties and a good strong masonry.

Another cause which brought destruction in many structure was the torsional effect. This was due to the difference between the center of rigidity and the center of mass (fig.2D). This produced the rupture of the small columns which were supporting the vault covering the amphitheater of the University in Bucharest. It is interesting to note that the rupture was stronger on the orthogonal directions of principal shear stresses (fig. 20 - 22).

With regard to the influence of foundation soils, there were made numerous interesting - even contradictory - statements.

Bucharest is situated on a slightly undulated plain, traversed by the Dîmbovitza river, whose main bed has 300-1000 metres breadth. The soil on the sides of the river has the characteristic features of loess and loess-clay layers. These strata are resting on a layer formed of gravel and sand, of great thickness. The bed of the river presents alluvial deposits consisting of fine sands with intercalations of mud and organic material. Therefore, the banks of the Dîmbovitza river are considered as having a weak foundation soil, with an allowable resistance of 1 - 1.50 kg/sq.cm. The higher region of the plain shows a more consolidated soil, with an allowable resistance of 2 to 3.00 kg/sq.cm.

Nevertheless, we shall mention that among the structures situated within the Dîmbovitza zone, only very few were damaged by the earthquake, and the damages were of slight importance. Even the churches - of a sufficiently old age - and generally sensible of earthquakes within this region behaved satisfactorily. On the contrary, most of the damages and destructions were noticed exactly in the region outward the Dîmbovitza banks, where the foundation soil, as mentioned, is much better. The level of the groundwater in the Dîmbovitza zone ranges from nearly 2 m depth in this region, whilst in the upper region it is met at a depth of 5 to 10 metres.

III. THE ANTISEISMIC COMPUTATION METHODS

The earthquake in Rumania of November 10,1940 imposed a total revision of the conception concerning structure designs when considering the destroying action of seismic motion.

In the first period after the earthquake, the principles of the Italian building laws for seismic regions were adopted, but the issued instructions for antiseismic building founded on these principles had no obligatory character.

After the war, on account of the great soaring concerning industrialization of the country and the increase of the construction of dwelling houses, the problem of antiseismic protection got the due importance. Therefore investigations were carried out to find the right method of computation and best measures of construction for designing quake resisting buildings.

We must admit that the mechanic phenomenon of the seismic shock not being satisfactorily studied, it cannot be introduced in the mathematical form necessary for designing of buildings and structures.

Indeed from mechanical standpoint, an earthquake can be reduced to a very complicated and irregular space motion of the earth crust. The seismic motion shows displacements with amplitudes, velocities and accelerations variable in direction and time. Even if all these elements could be registered on a special spot,as for instance at a seismical observatory, it is doubtful whether the registered values could be generalized for a whole region,in which the nature of soils, the stratigraphy, the underground water level might be quite different.

If the ground motion is known, the response of the foundation cannot be easily established and so much the more the motion of the building. The proper oscillations of the building have themselves an influence on the displacements of the foundation, so that it is very difficult to get a right idea of the stresses developped in the building. One must not forget that apart from the general movement of the building, each element of construction, walls, columns, girders, beams a.s.o. has its own vibration which can be the cause of great load stresses and can be a cause of destruction.

All this movements transmission forms what is called in new seismic terms "the interaction between structure and foundation ground".

On account of all these interactions the real pheno-

menon cannot be fixed in a precise mathematical form, and the necessity of simplification and approximation appears imperiously. In this form, the registration of the mechanical elements of the earthquake shocks was the first necessity. Recently new strong motion seismographs can furnish the necessary data as a basis for determining the earthquake intensity, but as these instruments are very expensive and cannot be installed in all seismic regions, other means of estimating the earthquake intensity are necessary.

The methods of estimating the earthquake intensity equivalent to its destructive force may be divided into two distinct groups, namely: subjective methods and methods based on the records of the real phenomenon (Richter-Gutenberg's magnitude, Housner's spectrum intensity a.s.o.).

The subjective methods use seismic scales which catalogue the earthquakes into several degrees (based on the impression of witnesses, the damage degree of structures a.s.o.) to each of them corresponding a conventional acceleration. The best known seismic scale is the modified Mercalli scale.

The magnitude and the spectrum analysis are based on records and they express the absorbtion energy of seismic waves. Contrary to the seismic scales, this mode of calculation of the seismic intensity takes no account of the subjective data about psychological expressions, damage of structures, whose behaviour depends on a series of factors which are not strictly depending on earthquake intensity, as for instance: inadequate designing and computation, improper materials, faults of execution, hidden defects etc.

As regards the calculation of building for seismic loads, two methods are known: the static method and the dynamic method. The static method neglects the elastic characteristics of structure considering - along the vertical - a linear variation of the seismic coefficient given by the Mercalli scale for the considered region.

The dynamic method considers the static-dynamic properties of the structure, by means of a seismic coefficient which varies along the height and depends on the elastic characteristics of the building. The earthquake characteristics may be estimated either by means of a seismic scale (see Soviet prescriptions) or by the seismic spectrum (see American recommendations). For the appreciation of the seismic forces, dynamic elements are used, but these forces, however, are considered in the end as statical forces.

The dynamic method has been developed throughout the last years by the American, the Japanese and the Soviet schools.

The American school studies the dynamic response of elastic structures reducing the latter to an equivalent system with a single mass. The influences of seismic disdurbances are introduced in form of a seismic spectrum whose variation is based on a corresponding calculation of the accelerations recorded during real earthquakes. The seismic forces of each story are determined by means of the fundamental shear force, the value of which is computed on the basis of the system with a single mass.

The Japanese code of 1955 estimates the seismic forces taking into account the elastic characteristics of structures, the type and degree of damping, the nature of foundation soil, and the seismicity degree of the region within which the structure is located.

The Soviet school considers structures as elastic systems with n degrees of freedom corresponding to the number of floors, and the seismic influence is assimilated by an analytical expression of the motion impressed on the foundation in form of a convergent series of trigonometrical functions. The seismic force is computed directly for the level of each story, the intensity of earthquake being expressed by coefficients according to the Mercalli-scale.

While the American recommendations consider a linear variation of the first normal vibration mode, the Soviet prescriptions distribute the loads along the height of the structure on the basis of the real accelerations of an elastic equivalent system.

Contrary to the American recommendations the Soviet prescriptions allow the calculation of seismic forces also in the case of superior vibration modes.

The application of the Soviet computation formula for seismic forces seems to be more rigorous from mathematic standpoint, while the American formula is practically simpler and more direct. It seems also that the earthquake effects by means of spectrum analysis approach the reality better than the assimilation of the motion produced by the seismic wave.

Although these methods are not yet perfect, they do represent an immense progress on the way of introducing dynamic methods into the design practice.

For both methods a still more difficult problem consists in precising the physical characteristics of structures and chiefly of the proper vibration periods, the seismic coefficients are depending on. There are some procedures to determine the periods but they don't respond completely to the

real physical phenomenon. Even "the exact computation methods" are only informative because the existing schemes introduce a series of symplifying computation hypothesis.

IV. CONCLUSIONS

It is very well known that the antiseismic design of buildings influences seriously the cost of many buildings. Now, when the economic problem is in the first rank all over the world, the choice of computation methods for antiseismic design approaching the most the reality is an utmost important problem. In this sense, the modern dynamic methods seem to be, at the present hour, best corresponding to the present needs.

In Rumania efforts were made to find the best means for antiseismic design of construction and in 1957 the project for a standard for the computation of seismic forces was elaborated. This standard is based on the dynamic methods and takes into account the soil characteristics, the interaction with the foundation, the elastic properties of the construction, the coefficient of seismicity of the region and the mass distribution along the height. On account of the lack of seismic records, the intensity of seismic motion is characterized by means of the Mercalli-Cancani-Sieberg scale.

But we must emphasize that, whatever the method used for antiseismic computation, on account of the simplifications and approximations and the uncertainty referring to mechanical effect as well as the elastic properties of the materials used in constructions, the method of computation and designing presents neither total accuracy nor complete certitude.

The recent earthquakes, among which that in Mexico City of July 28,1957, showed structures which, though complying with the modern computation prescriptions, presented more serious damages than structures which had not been designed according to the antiseismic prescriptions.

We think that the extremely complex phenomenon which accompanies an earthquake has not yet been sufficiently well studied in the investigations carried out so far.

We therefore consider that it would be extremely necessary to have more ample and various informations and to know the experiences of the different parts of the world, which would contribute to a judicious solution of thie problem.

Surely the first and the present Conference on engineering seismicity brings many new and very interesting views,

but it would be most profitable if a continuity could be created in the general studies of this delicate problem.

We suggest therefore the foundation of a permanent institution of international character called : "International Association for Engineering Seismology" having following objects in view :

a) The elaboration of an international intensity scale of the Mercalli type with the necessary instructions, which would allow a better and a more uniform estimation of the earthquakes intensity.

b) On the basis of this scale, the formulation of a universal questionary about the earthquake effects, in order to state the earthquake intensity on a more uniform basis.

c) To develop and set up a simple and cheap strong motion earthquake recorder - as for instance the Wilmot survey type - which should be installed in various observation points in high seismic regions.

d) To collect and centralize all informations, data, records and photographs about effects, damages and other characteristic elements of the earthquake.

e) To publish the important results obtained by the above mentioned means about different earthquakes as well as theoretical and practical studies of importance, regarding engineering seismology.

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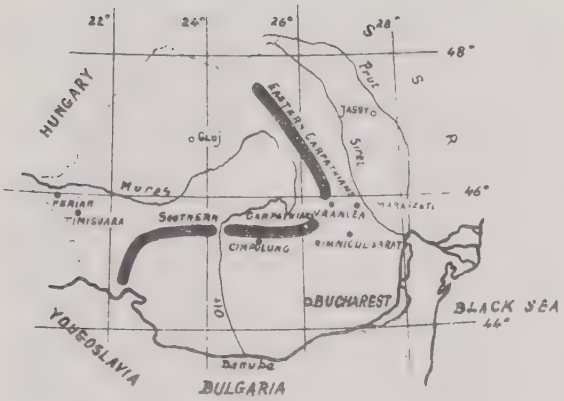

Fig. 1 Centers of seismicity

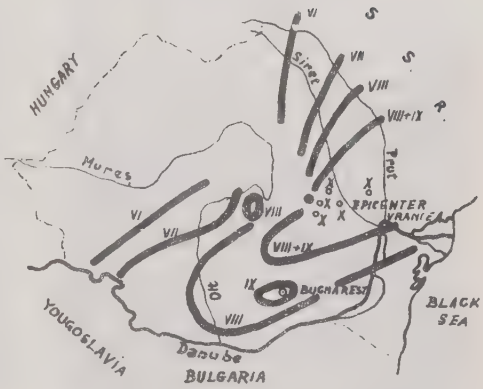

Fig. 2 Isoseistics of Rumania

Fig. 3 Building Carlton

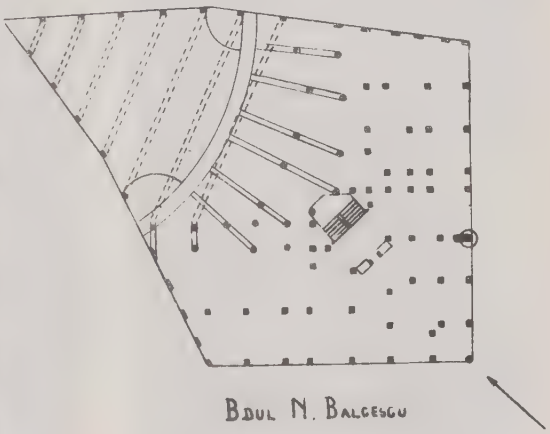

Fig. 4 Columns at the ground floor

Fig. 5 Building Carlton in fire

Fig. 6 Building Carlton
The column that collapsed

Fig. 7 Building Carlton
Detail of the collapsed column

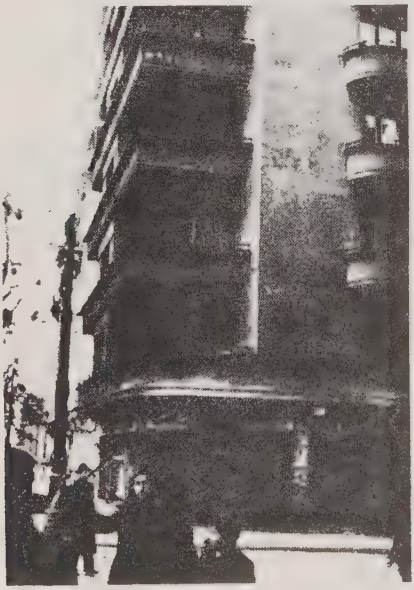

Fig. 8 Column with non corresponding bars

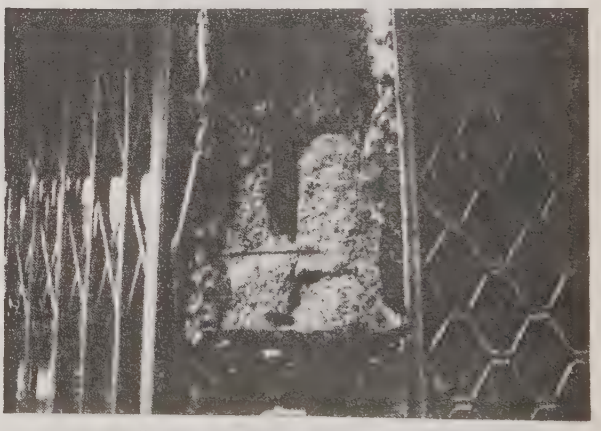

Fig. 9 Details for Fig. 8

Fig. 10 Too distanced ties

Fig. 11 Non corresponding structural joints

Fig. 12 Non corresponding columns

Fig. 13 Details of columns Fig. 12

Fig. 14 Non corresponding masonry

Fig. 15 Non corresponding masonry

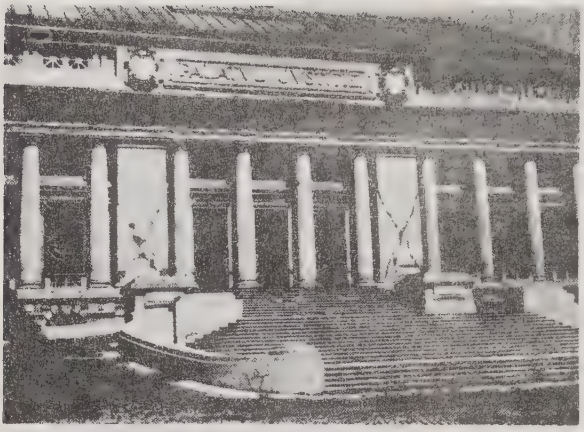

Fig. 16 X-shaped ruptures

Fig. 17 Non corresponding walls
Lack of ties

Fig. 18 Non corresponding gable

Fig. 19 Effects of gable Fig. 18

Fig. 20 Torsional effect

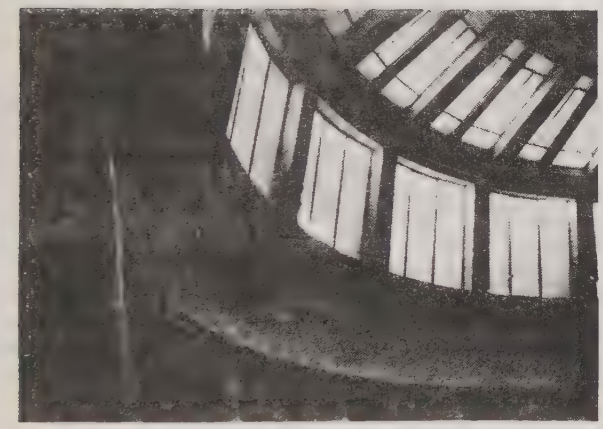

Fig. 21 University of Bucharest
Rupture of the cupota columns

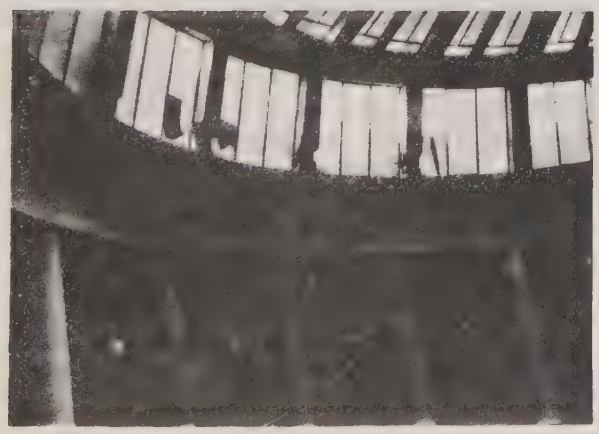

Fig. 22 University of Bucharest
Details of the columns of the cupota

ON THE RUMANIAN GENERAL DESIGN SPECIFICATIONS FOR CIVIL AND INDUSTRIAL BUILDINGS IN SEISMIC AREAS. EXAMPLES

by Em. Ţiţaru[x] and Al. Cişmigiu[xx].

Over an important area of Rumania, including the country's capital, earthquakes were recorded from time to time, the intensity of which attained the degrees 8...9 in the M.C.S. (Mercalli-Cancani-Sieberg) scale. The most severe earthquakes are rare and have a persistent hypocentrum located on the external part of the curvature of the Carpathian mountains at a depth of about 1oo...2oo km. Concern for preparation of aseismic design rules became actual only after the great earthquake of 1o.11.194o; this earthquake, whose magnitude according to the Richter scale was M = 7,4, caused important damage even in Bucharest, at an approximate distance of 170 km from the focus.

The preparation of aseismic structural design specifications remains even nowadays a difficult problem because design methods and especially the values of design parameters cannot be rigorously established. Our point of view was to give the practicing design engineer necessary indications in order to solve with maximum efficiency the cases of usual practice; such indications are based on an up-to-date knowledge of theoretical achievements as well as on design rules consecrated by existing experience in the construction and behaviour of aseismic structures. Thus some design data were established on basis of practice, of engineering judgement and of economic considerations.

1. DETERMINATION OF THE HORIZONTAL SEISMIC FORCES FOR THE STRUCTURE AS A WHOLE

1.1. In order to determine the seismic forces the theory of the "Response Spectrum" was adopted, because this theory was found to be at the present time the best fundamented both theoretically and experimentally answering at the same

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time to practical questions, (1),(2),(3),(4).

1.2. The equation adopted for the determination of the basic conventional shear force is:

$$Q_B = K.\beta.\psi.\varepsilon.P = \alpha.\psi.\varepsilon.P = c.P \qquad (1)$$

The factors introduced in this equation include all the elements of the response spectrum theory which influence the values of the seismic forces.

The parameter k establishes the seismic intensity of the site and depends on the characteristics of the focus, on the properties of the soil and on the distance to the focus. To the parameter β are ascertained values in direct proportion to the acceleration spectrum S_a, We consider that this factor expresses the highly complex soil-structure interaction.

The parameter ψ introduces the effect of internal damping in the structure.

The parameter ε is a factor by which the system with many degrees of freedom is made equivalent with a one mass system.

P is the resultant of all gravity permanent and live loads acting on the structure above the level from which deformation under action of seismic loads exists. The force P includes the weight of foundation if the deformability of the soil is considered in the determination of the dynamic elastic line of the building.

In the practical design equation, the product of the factors K and β is represented by the dynamic factor α .

1.3. Establishment of the factor k.

At the present time, it has become clear that the Mercalli scale and its various alternative proposals cannot serve as a basis for an engineering scale of forces. The establishment of a scale for Rumania has demanded an appreciation of an international scale of seismic intensities, corresponding to 4 zones:

Zone I: danger of moderate damage if buildings are built without respect of aseismic rules.

Zone II: danger of important damage.

Zone III: danger of very great damage.

Zone IV: danger of extremely grave damage. zones of maximum intensity for which adoption of general aseismic measures may be allowed.

The parameter k is typical for the relative intensity of the zones. It was accepted that k should vary according to a geometric sum. The ratio of this sum was determined on the base of a medium value of the basic shear force factor, accepted in engineering practice for the case of rigid masonry buildings sited on a medium ground; thus:

ε = o,oo5 for zone I;
ε = o,25 for zone IV.
It followed that $r = \sqrt[3]{5}$ and the k values: k_I = 1,o; k_{II} = 1,7; k_{III} = 2,9; k_{IV} = 5,o.

In a modern conception, the seismic zoning should be a "spectral zoning" aiming to give the design engineer the response spectrum determined by means of direct records made at the considered site. This being at present impossible, a practical solution is obtained by operations of "macro-zoning" and "micro-zoning", that is by combining general features with local ones. By "macro-zoning", a territory is divided into zones of probable seismic intensity, corresponding to the proposed scale. For appreciation of the intensity in the epicentral zone, use of Table 1 is recommended, which takes into account the depth of the focus and the magnitude of the earthquakes.

It is proposed that the seismic features of the soil should be considered in the seismic zoning as follows:

- in "macro-zoning", features of predominant basic layers should be considered (Table 1);
- in "micro-zoning" should be considered the characteristics of local surface layers.

In practice, the category of ground, to which the surface layer (on which the building is founded) belongs, is established by micro-zoning. This is of a qualitative nature. The quantitative aspect is introduced by the values of the factor , which differ according to the nature of layers. Thus the soil-structure interaction effect is pointed out for various categories of layers in the response spectra.

1.4. Determination of the parameter β.

This parameter is introduced in order to establish the standard response spectra corresponding to various soil conditions, in the case of a certain reference value "n" of the damping factor. The authors consider that existant experimental results as well as the behaviour of buildings in various soil conditions corroborate the existence of intervals of variation of predominant periods depending on the rigidity of the ground, as well as the possibility of a technical appreciation of the those conditions on the acceleration values.

We specially refer to the studies of the Japanese scientists Takahashi, Kanai, Kawasumi, (2), (3), to the behaviour of buildings in Mexico City during the earthquake of July 1957, to the Seattle spectra (Washington, USA) determined in 13 April 1949, (4), on soft grounds as compared with typical spectra for medium grounds, etc.

Grounds were classified in classes for which the following values of predominant periods were appreciated: very hard soils: o,15 s, hard soils: o,1...o,2 s, intermediate

soils: o,2...o,4 s, soft soils: o,3...o,6 s, very soft soils: o,6...1,2 sec.

The object of "micro-zoning" is to establish the categories of grounds, depending on surface layer stratification; this proceeds according to indications given in Tables 2 and 3.

The authors consider that the establishment of standard response spectra to be introduced in design specifications should follow this principle, the form and values of response spectra should result as medium forms and medium values of a great number of direct records of accelerograms on various categories of grounds, when making a survey programme of spectral zoning of a territory. For intermediate grounds the medium forms established by G.W.Housner (5) for Californian earthquake were adopted. These medium forms may satisfactorily be remplaced in the o,1...o,5 s interval by a straight line and in the o,5...3,interval by an equilateral hyperbole. For the remaining 4 categories of grounds, the response spectra were empirically drafted, always keeping in mind the general form of medium ground spectra, the behaviour of buildings in various ground conditions and the intervals of variation of predominant periods (fig.1). The forms of these spectra were considered to hold good for all four zones of seismic intensity outlined above. Any change in form of the spectrum as the distance from the focal zone varies was not considered, since, in the danger zone in which the earthquake may lead to damage, the predominant periods of the ground may be supposed to be constant, the variable factor being the ground acceleration (included in the parameter k).

In the interval T = o...o,1÷ o,15 seconds, it is proposed that the acceleration values should equal the double maximum value. This seems to be true, since for T = 0 the system must have the accelerations of the ground and very rigid buildings also posess a very small internal damping.

As in the process of seismic load determination it is possible to introduce the influence of internal damping, the equivalence of structures with many degrees of freedom with a one-mass system and the influence of higher modes of vibration, the spectrum variation was drafted up to 3 seconds, that is for the interval for which records exist.

The authors are thus persuaded that by a complex use of "macro-zoning", "micro-zoning" and of spectral forms differing by ground nature, light may be thrown on the following important practical issues:

a. distinguishing of seismicity of sites with different predominant basic layers,

b. distinguishing in a small zone with the same basic layer of the seismicity of sites with different surface layers.

We think it is unjustified to follow the practice of some

specifications which introduce the effect of soil on the spectra by changing only the intensity, without changing the form.

1.5. Establishment of the parameter ψ.

The influence of internal damping was not introduced by a system of spectral S_a curves, differing with the factor "n", but an independent parameter ψ was proposed. This parameter is defined as a ratio between ordinates of two spectral curves (S_v and S_a) for two different values of the factor "n", corresponding to the same period T.

Computing the "ψ" values for various "n" for the medium response spectra of intermediate soils, it results that these values are fairly constant for the entire scale of periods T. Thus it is possible to work in practice with a single stable "basic" curve computed for a "basic" factor "n", instead of considering a system of spectral curves corresponding to various "n". To pass from one spectral curve to another, corresponding to another "n" value, the basic curve is multiplied by factor "ψ". These results were considered valid for the spectral curves other ground categories too. The basic value of "n" was chosen to be $n = 20\% n_{cr}$, because this value corresponds to brick masonry buildings. With the medium values thus computed, the following analytical expression was established for the factor (fig.2):

$$\psi = \frac{2,92}{\sqrt{n}} \qquad (2)$$

It was considered that the factor "n" is not dependant on the vibration modes. The factor "n" is especially conditioned by the following important influences: the nature of building materials, the predominant strain, the intensity of the state of stress on the building as a whole, as shown in the "stress-strain" relationship, as regards great elasto-plastic or plastic strains. In table 4 the proposed values for ψ are given, depending on the following medium values admitted for "n": steel 2...6%, concrete and reinforced concrete 7...14%, brick masonry 15...25%, structural timber 1o...2o%.

In spite of the fact that present data are of a qualitative more than of a quantitative nature, such a proposal means to guide the design engineer in making a full and active use of the damping properties of various materials and structures and in the rational design of earthquake-resistant structures.

1.6. Determination of the equivalence factor ξ; the distribution law of seismic forces; the influence of higher modes.

The equivalence factor ξ of a system with many degrees of freedom with a one-mass system may be determined as

$$\varepsilon=\frac{\left[\sum_1^n P_i\cdot u_i\right]^2}{\left[\sum R\right]\left[\sum_1^n P_i\cdot u_i^2\right]}=\frac{\left[\sum_1^n P_i\cdot \xi_i\right]^2}{\left[\sum_1^n P_i\right]\left[\sum_1^n P_i\xi_i^2\right]} \quad (3) \text{ who } \xi_i=\frac{u_i}{u_n}$$

It may be shown that $\sum_1^n \varepsilon \to 1$

The conditions of equivalence in which this equation was established were: the identity of periods and the ecuality of energies. Considering the expressions of the inertial forces $F_i=\frac{4\pi^2}{T^2}\cdot m_i\cdot u_i$ a simple development leads to a distribution law of the basic shear Q_B along the height of the buildings:

$$S_i=\frac{P_i\cdot u_i}{\sum_1^n P_i u_i}\cdot Q_B=\frac{P_i\cdot \xi_i}{\sum_1^n P_i\cdot \xi_i}\cdot Q_B=\chi_i\cdot Q_B \quad (4)$$

Eq.(5) may be used in a direct determination of the seismic level shears S_i (without a preliminary computation of Q_B):

$$S_i=\left[\left(\alpha\cdot\psi\cdot\frac{\sum_1^n P_i\cdot u_i}{\sum_1^n P_i\cdot u_i^2}\right)u_i\right]\cdot P_i=\left[\left(\alpha\cdot\psi\cdot\frac{\sum_1^n P_i\cdot \xi_i}{\sum_1^n P_i\cdot \xi_i^2}\right)\cdot\xi_i\right]P_i=C_i\cdot P_i \quad (5)$$

The real displacement of the level (i) may be computed from eq.(6):

$$u_{i,real}=\frac{g}{4\pi^2}\cdot T^2\cdot C_i=24.87\,T^2\cdot C_i \quad (6)$$

The equivalence factor ε (eq.3) and the distribution law make possible a quick determination of seismic forces for higher vibration modes of structures with many degrees of freedom, using the response spectra for the one-mass system (fig.1), if the form of the dynamic elastic lines corresponding to the considered modes are previously computed.

In order to determine seismic forces for important prominences placed on buildings, the maximum values of forces which appear in higher vibration modes may be chosen and especially those which appear in the first vibration modes of the prominences.

2. DETERMINATION OF VERTICAL SEISMIC FORCES

The authors consider that the response spectrum theory may hold good for the vertical seismic action too. As the vertical rigidity of buildings is usually extremely great, the assumption was made that the vertical vibration period of the building is greatly influenced by the deformability of the ground. This leads practically to short periods, situated on the constant part of the spectrum, that is to maximum values for accelerations. In the special case of big span cantilevers and roofs, if the vertical flexibility of the structure becomes important, the periods increase and the influence of higher modes may be considered. In analysis, equations (1), (3), (4) hold good, but for the factor γ, for

which values amplified from 3 to 4 times were proposed.

3. DETERMINATION OF SEISMIC PRESSURES EXERTED BY LIQUIDS AND FRIABLE MASSES IN SILOS

The seismic pressures p_0 exerted by liquids stored in reservoirs are considered to be constant along the height of the vessel and are determined by eq.(7)(fig. 3):

$$p_0 = \omega \cdot \alpha_{max} \cdot \delta \cdot H \qquad (7)$$

The seismic pressures of friable masses in silos result from the increase in active thrust due to the diminution of friction angles, to which the resultant of inertial forces corresponding to the stored mass is added.

* * *

With the values of ε and T computed for the 1-st mode, the ratio $\rho_1 = \frac{\varepsilon_1}{T_1}$ may be evaluated by means of which the necessity of analysing higher modes may be appreciated. The ratio ρ may serve as a criterion of dynamic rigidity, more satisfactory than the classifications made on basis of the period of the first mode only. A curve $\varepsilon = \varepsilon(T)$ (fig.4) may be traced for the ε, T values of various modes; this curve may help to forecast approximately the ε, T values for higher modes, using the ε, T, values computed for the lower modes; this may lead to important advantages in practical analysis.

4. ASEISMIC STRUCTURAL ANALYSIS

4.1. Principles

The chapter of aseismic structural analysis was developed on the following principles:

1. In problems of strength of materials (general sectional stresses, sectional stresses on structural members, unit stresses), rigidity (relative level rigidity, general rigidity, rigidity of structural members), stability (mechanical and elastic, - local and general), vibrations (determination of periods and of vibration modes), consideration must always be given to the spatial co-operation of structural members.

2. The behaviour of the building must be considered not only in the stage in which the prescribed forces may act, but for all phases, ranging from 0 seismic load up to the collapse load, and following step by step the process of development of elastic and plastic strain, during the dynamic adaptation phenomena.

3. An increase in strength and stability of the spatial structure must be achieved by alloting some strength reserves in members and sections of decisive rôle.

4.2. Basic principle of the dynamic-spatial analysis

The authors developed a general method for the dynamic

and spatial structural analysis of buildings stressed by earthquake loads. This method comprises all the steps needed in an exhaustive analysis, such as:

1. the determination of dynamic elastic lines;
2. the determination of periods;
3. the determination of the basic shear;
4. the distribution of the seismic forces along the height of the building;
5. the determination of seismic forces acting on the principal resistant members of the spatial structure (frames, diaphragms, special bars, etc.);
6. the determination of sectional stresses in various sections of the structural members.

The method may be applied for any vibration mode. The fundamental idea of the method lies in the fact that it coordinates in a comprehensive method the spatial behaviour, the active use of principles outlined in 4.1, of the response spectrum theory (eq.1, 4, 5) and of free vibration analysis (periods and forms).

The consideration of spatial co-operation for the determination of displacements in the structure as well as for the determination of the loads acting on every resistant member (frames, vertical and horizontal diaphragms, special bars) was achieved by making use of the generalised method of the centre of rigidity, the basic notion of which is the relative level rigidity. This may be defined as the shear force at level (i), corresponding to a unit relative displacement: (6) $R_i = \frac{Q_i}{\Delta_i}$ (fig.5). The quantity R_i is not a constant for the considered member at the considered level (i), since it depends on the general elastic line of the member (variation of rigidity and the distribution of forces along height); R_i may even change of sign ($\pm$) along the height of the building. It must be stressed that the expression for R_i has significance only if the state of strain is caused by bending with shear.

In the special case when on a certain length pure bending ($Q_i=0$) exists, an indetermination of the form $\frac{0}{0}$ appears, which may be solved by correcting the real shear force diagram by a supplementary force acting at the top of the member. This force, of small intensity compared with the other forces (thus slightly influencing the displacements) does not allow the shear to vanish along the height of the considered member.

From the practical point of view, the sensibility of R_i (that is the variation of R_i with the variation of the load diagram of the considered member) may be considered as follows:

1. In the case of short cantilever diaphragms, when bending strain may be negligible, R_i is invariable and becomes a geometric constant;
2. in the case of frames in which displacements due to shears are predominant, the sensibility of R_i is small;
3. in the case of "bars" in which bending strains are

dominant or cannot be neglected, R_i has an extreme sensibility to small variations in the loading diagram.
If the principal inertial axes of some vertical members are not parallel to two orthogonal axes, it will be necessary to determine for each member the oblique relative level rigidities and the principal axes of rigidity at level (i), (7), always keeping in mind the above-mentioned definition of R_i. By the generalised expression of the relative level rigidity and its properties in the form above (dependency on, and sensibility in, the variation of rigidities and masses along the height of the member, the property of being alloted a sign, possibility of solving indeterminations on portions stressed in pure bending, the consideration of oblique rigidities) a basic criterion is obtained for the establishment of the compatibility of strains along the height of the member and in the plane of the horizontal diaphragms. This criterion permits the solving of the problems of establishment of the elastic lines and the distribution of forces for any loading system acting on a spatial structure; that is the solving of the problem of the higher vibration modes. The method may easily be suited to step-by-step procedures.

Starting from the method proposed by L. Grinter for the analysis of frames acted on by lateral forces (8), the authors have outlined in Table 5 the steps necessary to obtain the relative level rigidities, the elastic lines, and the solution for lateral forces, in a practical form suitable to step-by-step procedures. This table may also be used in the determination of periods and vibration forms of independent frames.

In the analysis of diaphragms with apertures, the authors use the method of forces, with a grouping of redundants (fig. 6) and certain simplifications due to practical considerations.

In the method outlined above, the spatial aseismic structure is treated as a "complex bar" supported by a deformable medium; this was achieved by a generalisation of the notion of "simple bar".

Regarding the dynamic analysis (the determination of forms and periods of vibrations), the energy method with a step-by-step procedure is adopted, for all vibration modes.

The dynamic elastic line corresponding to a certain vibration mode is the elastic line of "the complex bar" under action of inertial forces in direct proportion to the seismic forces computed by eq.4. Thus the sum of the conventional inertial forces (with which the vibration form was determined) results in a basic shear Q'_B , "m" times bigger than Q_B (eq.1). This result permits the coupling of the dynamic analysis for the determination of vibration forms and periods with the determination of seismic forces on the base of the response spectrum theory (chapter 1).

The step-by-step procedure for the 1-st mode.

I-st Approximation. Step I_1. Determine in a preliminary approximation the relative level rigidities $R_{i,1,0}$ for each vertical resistant member, for a horizontal loading diagram in proportion with the gravity level forces P_i.

Step I_2. The loading of the spatial structure with the horizontal inertial forces $F_{i,1,I} = P_i$ and the determination of sectional level shears $Q_{i,1,I}$.

Step I_3. The distribution of $Q_{i,1,I}$ to the resistant members, on the base of the relative level rigidities $R_{i,1,0}$.

Step I_4. Determination of a new approximation of the relative level rigidities, $R_{i,1,I}$ on the base of forces obtained in step I_3.

Step I_5. Determination of relative level displacements $\Delta_{i,1,I}$, of the absolute displacements $u_{i,1,I}$ and of those relative of the elastic axis the spatial structure with the eq.:

$$\Delta_{i,1,I}=\frac{Q_{i,1,I}}{\Sigma R_{i,1,I}} \; ; \quad u_{i,1,I}= u_{i-1,1,I}+\Delta_{i,1,I} \; ; \quad \xi_{i,1,I}=\frac{u_{i,1,I}}{u_{n,1,I}}$$

Step I_6. The determination of the periods in the approximation I:

$$T_{1,I} = 0{,}2 \sqrt{\frac{\sum_1^n P_i \cdot \xi_{i,1,I}^2}{\sum_1^n P_i \cdot \xi_{i,1,I}}} \cdot \sqrt{u_{n,1,I}}$$

in which $u_{n,1,I}$ is the top displacement in cm.

II-nd Approximation

Step II_1. The spatial structure is loaded with the forces $F_{i,1,II} = P_i \cdot \xi_{i,1,I}$, and the $Q_{i,1,II}$ is drafted.

Step II_2. The distribution of $Q_{i,1,II}$ to the vertical resistant members, depending on $R_{i,1,I}$.

Step II_3. The determination of a new approximation of the relative level rigidities $R_{i,1,II}$.

Step II_4. The determination of $\Delta_{i,1,II}$; $u_{i,1,II}$ and $\xi_{i,1,II}$ on basis of eq. determined in step I_5.

Step II_5. The determination of the period in the approximation II.

$$T_{1,II} = 0{,}2 \sqrt{\frac{\Sigma P_i \cdot \xi_{i,1,I}^2}{\Sigma P_i \cdot \xi_{i,1,II}}} \cdot \sqrt{u_{n,1,II}}$$

Approximation nr. N

Step N_1. The steps are to be continued till the relative elastic line is the approximation N-1 and N fairly coincide:

$$\xi_{i,1,N} \sim \xi_{i,1,N-1}$$

Step N_2. The period and the equivalent factor for the 1-st mode are:

$$T_{1,N}=0{,}2\sqrt{u_{n,1,N}} \; ; \quad \varepsilon_{1,N} = \frac{\left[\sum_1^n P_i \cdot \xi_{i,1,N}\right]^2}{\left[\sum_1^n P_i \cdot \xi_{i,1,N}^2\right]\left[\Sigma P_i\right]}$$

Step N_3. The shear at the base is computed for the conventional forces:

$$F_{i,1,N} = P_i \cdot \xi_{i,1,N-1}$$

$$Q'_{B,1,N} = \sum_1^n P_i \cdot \xi_{i,1,N-1}$$

Step N_4. The basic seismic shear force for the 1-st mode $Q_{B,1,N}$ is computed with eq.1, for $T_{1,N}$.

Step N_5. The factor $m = \alpha_{B,1N}/\alpha_{B,1N}$ is to be determined.

Step N_6. Determination of seismic level forces with the eq.:

$$Si = m.Fi,1,N = m.Pi.\xi_{i,1,N-1}$$

It is thus sufficient to amplify by factor "m" the shear force diagrams obtained by distribution of sectional shears in approximation N, for each vertical resistant member.

Higher modes of vibration

The succession of steps is, in principles, the same as for the 1-st mode. As a first approximation for the form of mode j , the statical elastic line under gravity loads P_i may be taken, considering j -1 intermediate supports at the nodes of the considered mode, the position of which are chosen by experience.

It is known that the tentative analysis of higher forms of vibration by applying simply step-by-step procedures is bound to be unsuccessful, because it always leads to the form of the first mode. But, if the orthogonalizing condition of forms and the "purifiying" conditions are repeated at every step, the forms of lower modes are eliminated and the chosen form is obtained. Lack of space does not permit a more developed discussion.

As shown above, the frequent case of a structure with two rectangular axes of symmetry (for which a longitudinal and a transverse analysis is made) may be solved correctly. The method may be extended for the case of assymetric structures if the displacements (rotations and linear displacements in a plane perpendicular to the direction of earthquake) of the axis of the structure are small compared with the displacements of the real spatial axis projected in the plane of seismic action.

4.3. On dynamic adaptation.

The authors recommended in the Draft Specifications the consideration of the property of adaptation of structures to seismic action. We think that the notion of "adaptation", set forth by A.Caquot (6) for the case of a system of forces of constant form and value, may be extended for the case of dynamic seismic action. This is justified by the limited duration of alternative seismic stresses and by the experience of damage processes in buildings, showing a stress redistribution. The conception of analysis based on dynamic adaptation is best suited for aseismic design as it permits a correct evaluation of the safety of structures, an improvement in general behaviour and an economically advantageous design. Making use of the property of adaptation, the effects of diminution of seismic forces and of stress redis-

tribution between members of the spatial structure may be considered, according to behaviour schemes of the structures in limit stages of stress and strain. To apply actively seismic adaptability, particular judgement is needed in directing the process of plastification (positions and succession of plastic hinges and zones). The section in which plastic strain occurs for one or more sectional stresses (bending moment, shear force, torsional moment, tensile or compressive normal force) should be designed to resist safely to the rest of sectional stresses. A way for future investigation in aseismic structural analysis, outlining the capacity for seismic adaptation, is set forth by G.W.Housner's proposal (10). The condition of stability of a limit stage is: the sum of elastic and plastic strain energies for which a structure is capable in the considered limit stage should exceed the total seismic kinetic energy E_c allotted to the structure. The parameters set forth in chapter 1 permit evaluation of E_c in eq.(8):

$$E_c = \frac{1}{2} \cdot M \cdot S_v^2 = \frac{P}{2g} \cdot \left(\frac{\alpha \cdot \psi \cdot T}{2\pi} \right)^2 \qquad (8)$$

For more than 10 years, limit design specifications are in use in our country for the design of sections in the ultimate stage and for the analysis of some structural members in which stress redistribution due to plastic strain is allowed for, in the case of statical forces. At the present time, studies and investigations are made for the generalisation of limit analysis and design to all building materials. Such Specifications allow for the practical use of seismic adaptability and of the energy analysis outlined above. In our Draft Specifications, use of the property of dynamic adaptability on basis of assumptions of probable damage is especially recommended in the case of buildings with masonry panels and those with load-bearing walls.

4.4. Creation of strength reserves.

Creation of strength reserves is recommended in the following cases:

a. when uncertainties subsist in the appreciation of spatial co-operation of some members of the structure or regarding stress flow;

b. when reserves exist in section due to constructive arrangements, which may become seismically effective with no supplementary uneconomic expense of materials and labour;

c. when uncertainties subsist in seismic load determination.

4.5. The aseismic analysis of multi-storey buildings.

The advantages of the method proposed above are especially pointed out in the aseismic analysis of high multi-storey structures performed by the response spectrum theory. Regarding the damping, the function $\psi=\psi(n)$ allows for a fairly correct appreciation on basis of experimental data, of the damping factor and its influence on the spectrum. It is thus possible to distinguish between various high buildings, regarding the building materials used in the structure and cladding as well as the structural solution itself (frames, diaphragms, rigid or flexible structures, etc.).

The function $\mathcal{E}=\mathcal{E}(P_i,\mu_i)$ allows for the exact determination of the equivalence factor for any vibration mode, any mass distribution and any variation of rigidity. The law of distribution of seismic forces (eq.4) is quite general and takes exactly into account the form of the elastic line.

The method of dynamic and spatial analysis takes also into account the spatial co-operation of all members resistant to lateral action (including the consideration of the rigidity of resistant masonry panels). The higher vibration mode can be completely and exactly determined by the energy method using step-by-step procedures, with a "purification" of the influence of lower modes at each stage. Thus, the use of response spectre up to 3 seconds appear to be justified.

5. ASEISMIC CONFORMATION

The chapter regarding "Aseismic conformation" was developed in such a manner as to give the design engineer the necessary elements for a rational and active organisation of strength rigidity and stability in the designed structure.

Such elements put forward the complex interaction between choice of structure and of building materials and the optimum stress flow under lateral earthquake action.

The designer's creative thought, rather not acquainted with the seismic action, is systematically directed to the pursuit and understanding of the behaviour of the structure as an integrated whole, from „0" seismic load up to the collapse load, in order to set forth any adaptation possibilities by material and by structure, to achieve thus an optimum solution.

This chapter develops the following main problems:

1 - General rules of aseismic building conformation
2 - Arrangement of masses and resistant members.
3 - Arrangement of aseismic joints
4 - Conformation of foundations
5 - Conformation of vertical resistant members (frames and diaphragms)
6 - Conformation of transverse ties between vertical resistant members.
7 - Aseismic conformation of non-structural members (principal and secondary).

8 - Aseismic conformation of important prominences
9 - Aseismic conformation of buildings made of various materials (timber, steel, reinforced concrete cast "in situ" and precast masonry).
10 - Aseismic conformation of various types of structural (industrial halls, multi-storey buildings).

A somewhat larger development was given to buidings made of precast parts, because the Economic Development Plan of the Rumanian Republic lay particular stress on the industrialisation of the constructional activity by prefabrication (12).

Regarding the answer to the question "Which structures are to preffered in seismic zones, rigid or flexible ones ?" the authors' point of view, based on their engineering practice is outlined below:

a. flexible structures should be preffered in the case of light structures, in which danger of damage in wall panelling is excluded, due to a correct arrangement (wall panelling should not be dragged along in the deformation of the structure);

b. rigid structures are of advantage in the case of buildings made of traditional materials and in the case of monumental buildings.

6. EXAMPLES

The application of the Draft Specifications as well as studies and design of aseismic buildings carried out by the authors have contributed towards the development in the Rumanian Peoples' Republic of a modern conception in aseismic design.

In fig. 7,8,9,1o,11,12,13 some typical structures built in Bucharest are shown, which were directly designed by the authors or for which the authors served as Consulting Engineers for aseismic design. The designs were carried out in the principal Design Offices in Bucharest:

I.P.C.M.C. - Design Office for Structural Engineering;
I.P.B. - Design Office for the City of Bucharest;
I.S.C.A.S. - Design Office for Studies in Construction Town Planning and Architecture.

FIGURE CAPTIONS

Fig.1. Response spectra for various ground categories and various seismic zones.
Fig.2. Diagram of the damping function ψ
Fig.3. The ω diagram.
Fig.4. Relation curve between the equivalence factor $\mathcal{E}$ and the natural vibration periods T for various modes.
Fig.5. The relative level rigidity.
Fig.6. The analysis scheme of diaphragms with apertures.
Fig.7. Typical 8 storey block, with rare aseismic reinforced concrete walls cast "in situ", of "cellular" type (Co-workers: D.Badea and T.Popp, Struct.Engrs.)

Fig. 8. Ibidem (Co-workers: D.Badea and Gr. Eliescu, Struct. Engrs.).

Fig. 9. 8 storey block with flexible ground-floor, in reinforced cast "in situ" concrete (co-workers: T.Popp and D.Moscuna, Struct. Engrs.).

Fig. 10. Typical 8 storey block in precast reinforced concrete, "big panel" system (Co-workers S.Cristescu, N.Mihalache, A.Sorbală, Struct.Engrs.).

Fig. 11. 14 storey block, of "cellular" type with aseismic reinforced concrete cast "in situ" walls (Co-workers, D.Badea, Gr. Eliescu Struct.Engrs.).

Fig. 12. Tower buildings 18 storeys high in reinforced cast "in situ" concrete. The complex structure is an example of a complete application of the analysis method described in chapter 4.2.
(Chief Design,Enginer P.Vernescu, Struct.Eng. coworker Z.Măduţă, Struct. Eng.).

Fig. 13. Tower building 18 storeys high. Project, Structure composed of a central reinforced concrete "tower" cast "in situ" and lift slabs. (Co-worker, I.Berleanu, Struct.Eng.).

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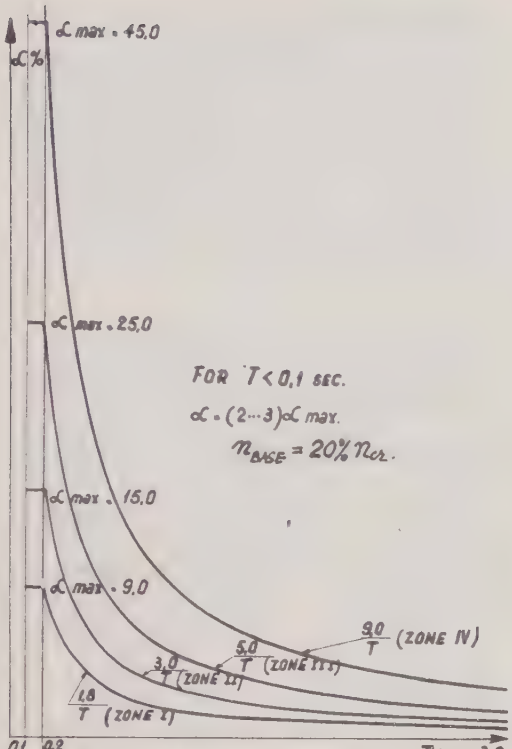


FIG. 1a VERY HARD GROUND

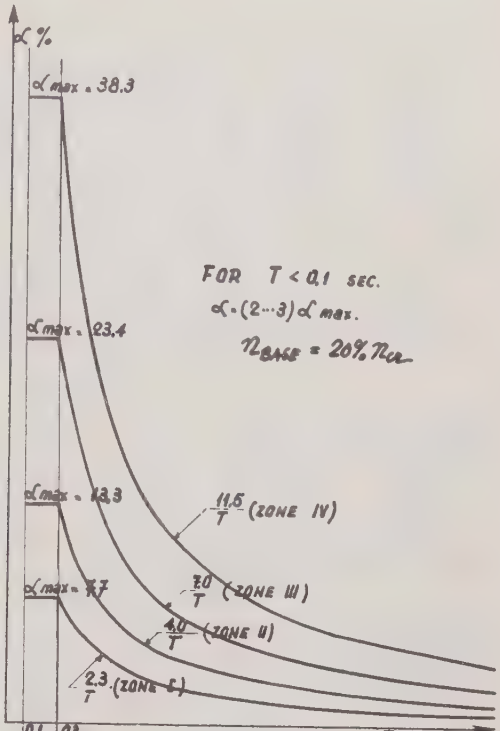


FIG. 1b HARD GROUND

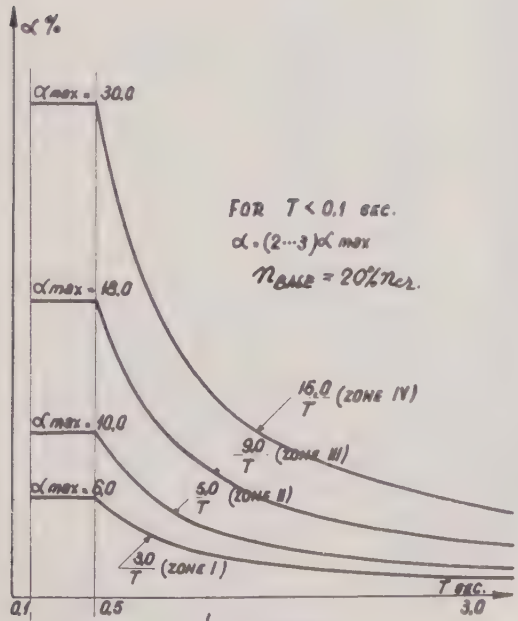


FIG. 1c. FIRM GROUND

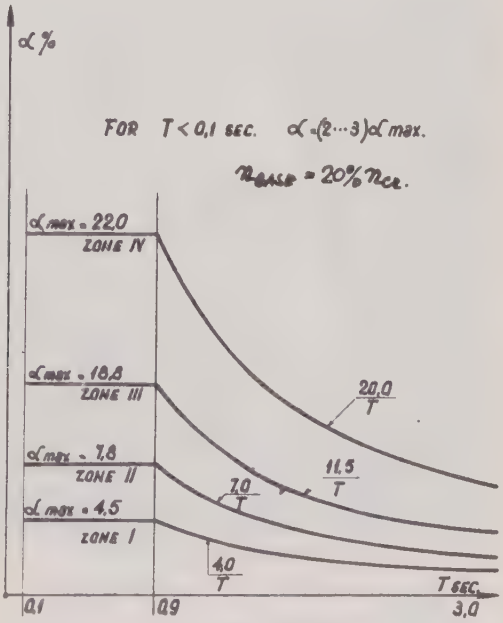


FIG. 1d. SOFT GROUND

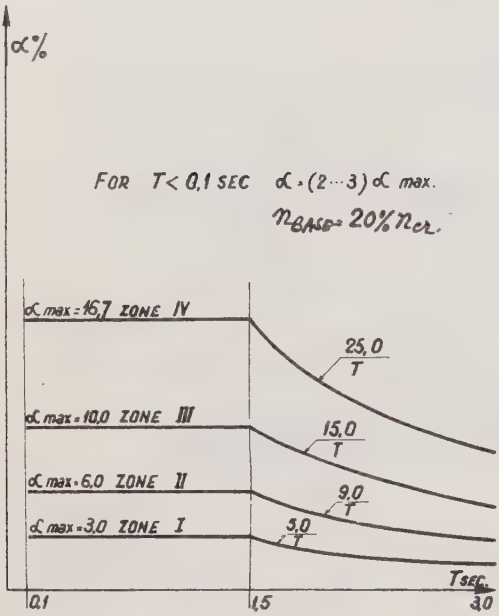


FIG. 1e VERY SOFT GROUND

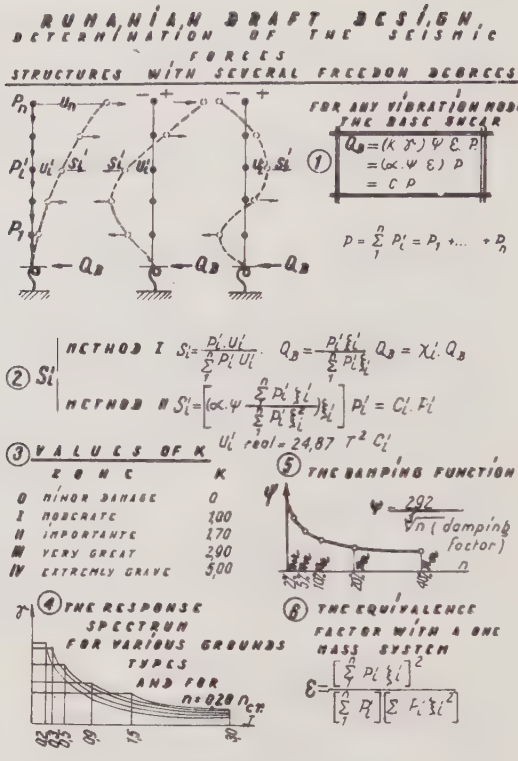

RUMANIAN DRAFT DESIGN
DETERMINATION OF THE SEISMIC FORCES
STRUCTURES WITH SEVERAL FREEDOM DEGREES
FOR ANY VIBRATION MODE THE BASE SHEAR
Q_B = (k β) ψ ε P = (α ψ ε) P = C P
P = Σ P_i' = P_1 + ... + P_n
METHOD I
METHOD II
VALUES OF K
ZONE K
0 MINOR DAMAGE 0
I MODERATE 1,00
II IMPORTANTE 1,70
III VERY GREAT 2,90
IV EXTREMLY GRAVE 5,00
THE DAMPING FUNCTION
THE RESPONSE SPECTRUM FOR VARIOUS GROUNDS TYPES AND FOR n = 0,20 n cr
THE EQUIVALENCE FACTOR WITH A ONE MASS SYSTEM

FIG. 1f.

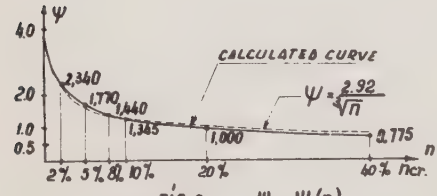


FIG. 2 ψ = ψ(n)

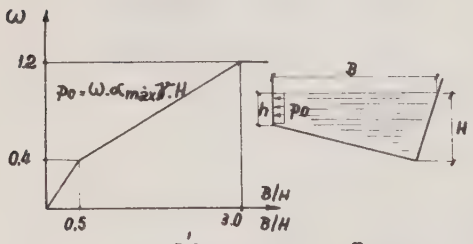


FIG. 3.

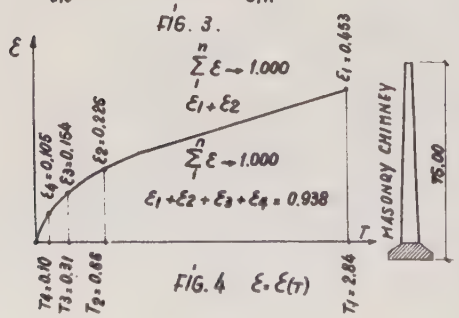


FIG. 4 ε. ε(T)

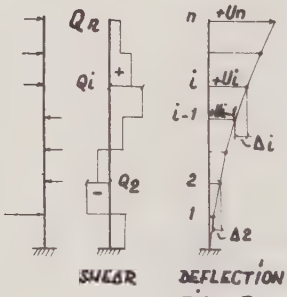


FIG. 5

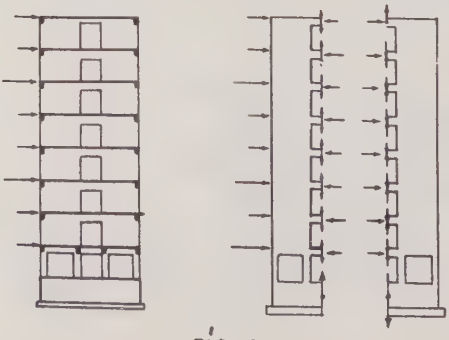


FIG. 6

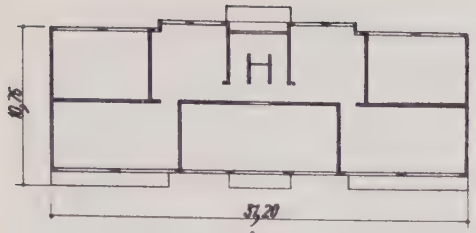


FIG. 7

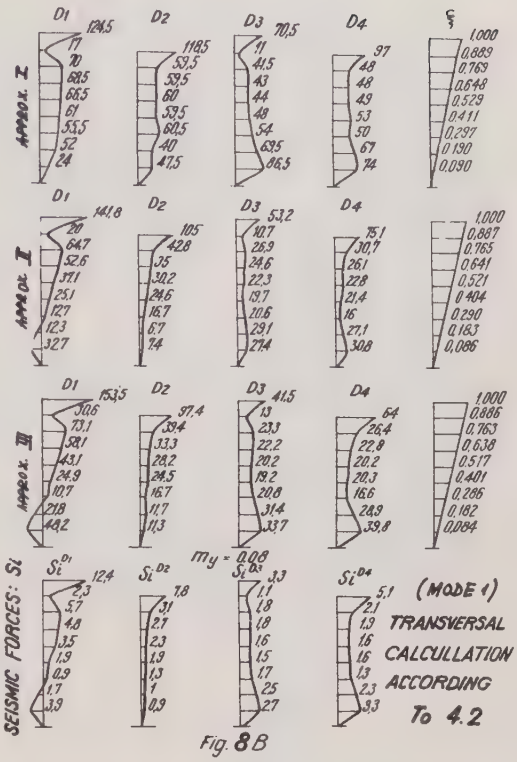


Fig. 8B

RESULTS

x - x

$T_x = 0.675$ $\alpha = 0.0444$

$\varepsilon = 0.857$ $\Psi = 1.3$

$Q_B^x = 0.039\,D = 136^t$

$Q_B^m = 1437^t$

$m_x = \frac{Q_B^x}{Q_B^m} = 0.093$

y - y

$T_y = 0.71$ sec. $\alpha = 0.0423$

$\varepsilon = 0.772$ $\Psi = 1.3$

$Q_B^y = 0.0425\,D = 149^t$

$Q_B^m = 1863^t$

$m_y = \frac{Q_B^y}{Q_B^m} = 0.08$

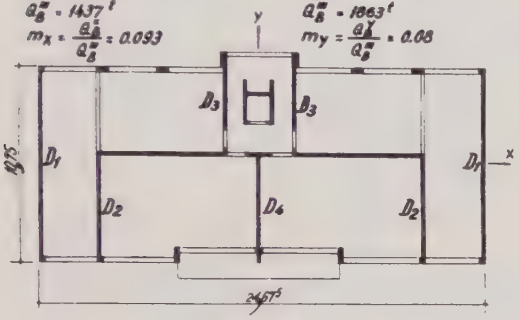


Fig. 8a

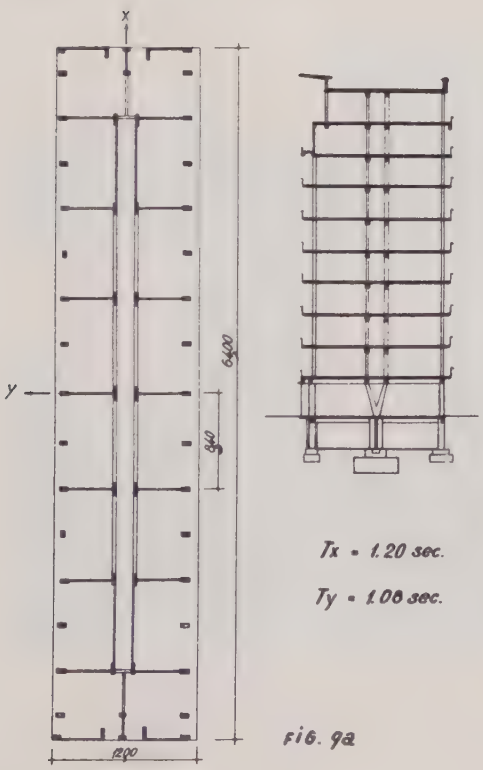


fig. 9a

$T_x = 1.20$ sec.

$T_y = 1.06$ sec.

Fig. 9b I.P.B.

Arh E COSMATU

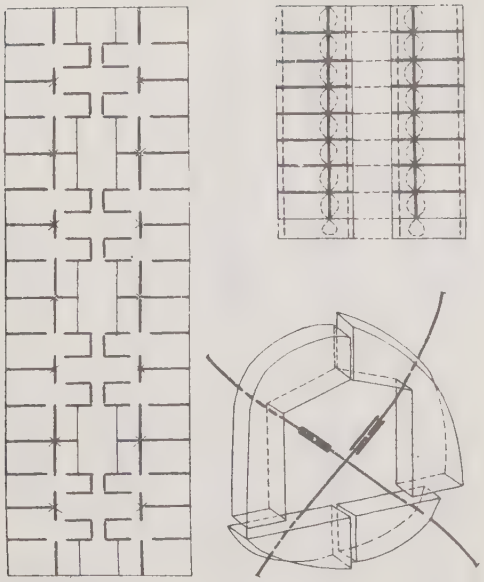
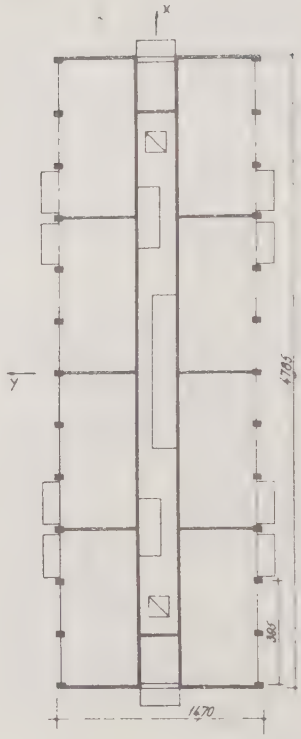


Fig. 10



$T_x = 0.5$ sec.
$\alpha = \frac{0.03}{T} = 0.06$
$\Psi = 1.25$
$\varepsilon = 0.77$
$Q_B^x = 0.058\ D$

$T_y = 1.20$ sec.
$\alpha = \frac{0.03}{T} = 0.025$
$\Psi = 1.35$
$\varepsilon = 0.75$
$Q_B^y = 0.025\ D$

Fig. 11a

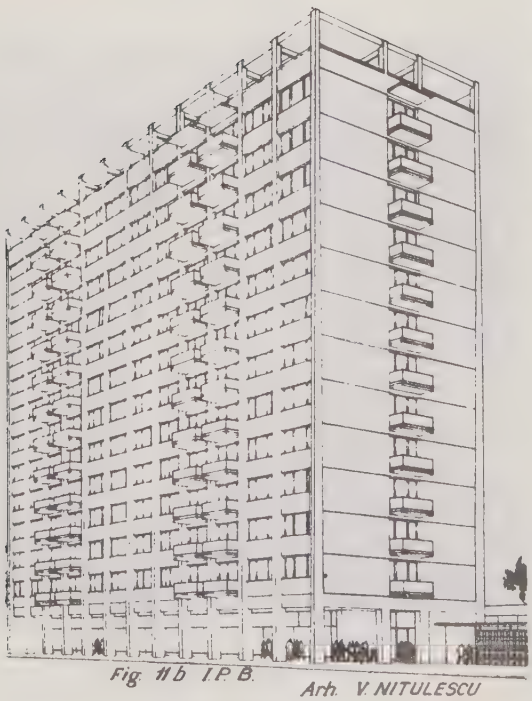
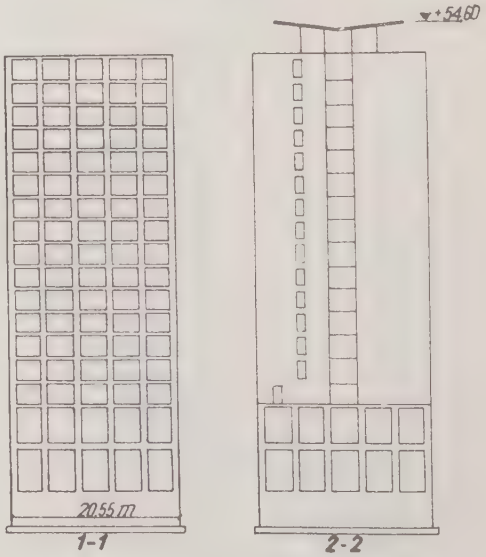
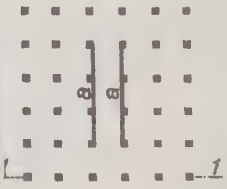


Fig. 11b I.P.B. Arh. V. NITULESCU





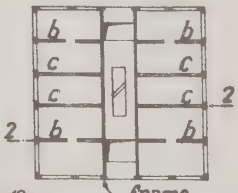


Fig. 12a

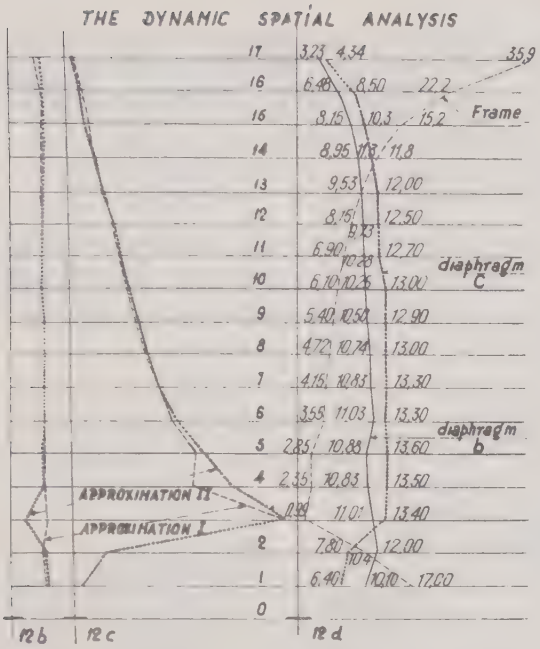


Fig. 12b THE RELATIVE LEVEL RIGIDITIES FOR FRAME.
Fig. 12c THE RELATIVE LEVEL RIGIDITIES FOR DIAPHR. b
Fig. 12d THE DISTRIBUTION OF SHEARS FORCES.

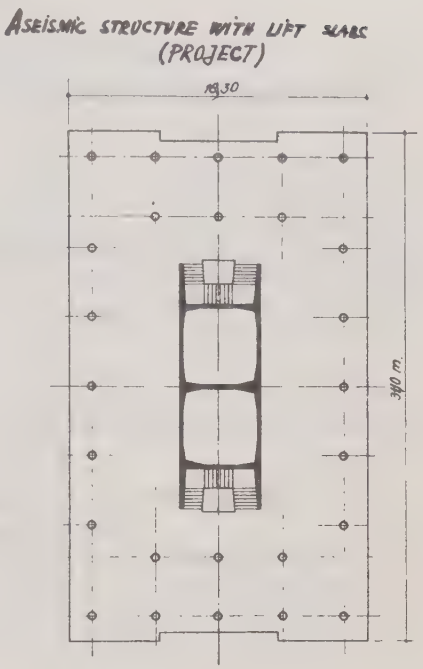


FIG. 13a

Fig. 12e

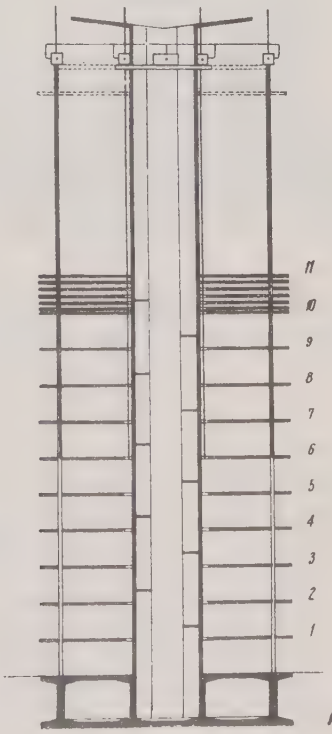


FIG. 13b

Table 1

Zone	Surface focus		Intermediate focus	
	H=5...10 km	H=20 km	H=40...60 km	H=100...200 km
I	4,5	4,5...5,3	5,3...5,9	6,0... 6,9
II	4,5... 5,3	5,3...5,9	6,0...6,9	7,0... 7,7
III	5,3... 5,9	6,0...6,9	7,0...7,7	7,75... 8,5
IV	6,0... 6,9	7,0...7,7	7,7...8,5	8,5
IV	7,0... 7,7	7,75..8,5	8,5	

Note. In zones of deposits, it is appreciated that the intensity may increase by 1...2 steps, as compared with rocky zones at the same distance from the epicentral zones.

Table 4

Material of construction	Shear strain due to shear stress predominant	Strain due to normal stress predominant	
	Rigid diaphragms	Frames, trusses, flexible diaphragms	Slender flexible structures, chimney-stacks, radio masts, columns
Brick masonry	1,00	-	1,35
Concrete and R.C,	1,20	1,35	1,50
Steel	1,35	1,50	1,75
Timber	1,00	1,35	1,35
Massives structures	2,00	-	-

Note. In cases in which the procentual critical damping "n" can be appreciated from well established data, the factor should be computed by

Table 2

Classification of ground layers

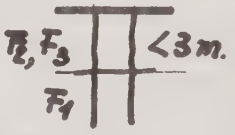
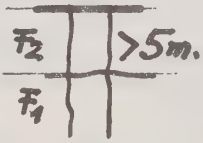
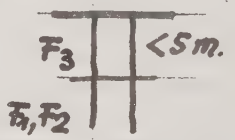
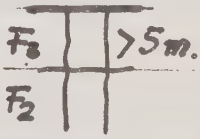
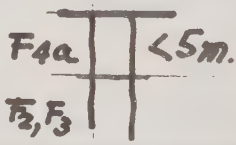
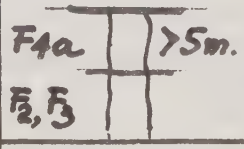
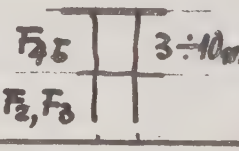
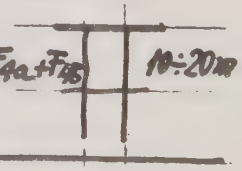
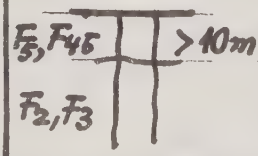
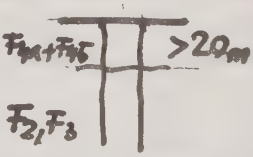
Type	Description
F_1	Formations extending on large areas, of early tertiary or pro-tertiary age, composed of rocky eruptive sedimentalyer metamorphic layers, - massive, compact, - with very high allowable pressures ($\sigma < 10$ kg/cm^2).
F_2	Early tertiary formations or dilluvial deposits such as - marl rocks, - cemented sands, - sand - gravels, compact, of very small compressibility and high allowable pressures ($4 < \sigma < 10$ kg/cm^2), regardless of humidity ratio.
F_3	Dilluvial or alluvial formations of medium allowable pressures ($2 < \sigma < 5$ kg/cm^2) such as: - sandy grounds (of medium compressibility or compact), regardless of humidity: - clayey grounds (clays, sandy clays, clayey sands) dry or humid, with a consistency ratio $I_c > 0{,}75$, of medium compressibility; - sandy gravels - big-pore grounds dry or humid, of eolian or other origin
F_4	a. Alluvial formations, dry or humid, compressible, with low allowable pressures ($0{,}8$ kg/cm$^2 < \sigma < 2$ kg/cm^2) consisting of: - fine or dusty sandy soils, with or without gravel interstices; - clayey soils of plastic consistency; (I_c - $0{,}50$.. ..$0{,}75$) (dusts and clys) b. Ibidem, under water level.
F_5	Very recent fine alluvial formations, saturated, very compressible, unconsolidated with small allowable pressures ($\sigma < 0{,}8$ kg/cm^2) consisting of: - fine dusty fluid sands; - soft clayey grounds, fluid; - marshy soils (bogs) typical for Delta grounds for lake bottoms and for the inferiour part of certain siver beds; - macroporous grounds, inundated.

Notes. 1. "Alluvial formations" are formations made of boggy deposits, of sand, gravel and rocks, carried and deposited by water transport during and after the quaternary age.
2. "Dilluvial formations" are unconsolidated rocky formations on slopes or at base of slopes, deposited during and after the quaternary age, by accumulation of material destroyed by

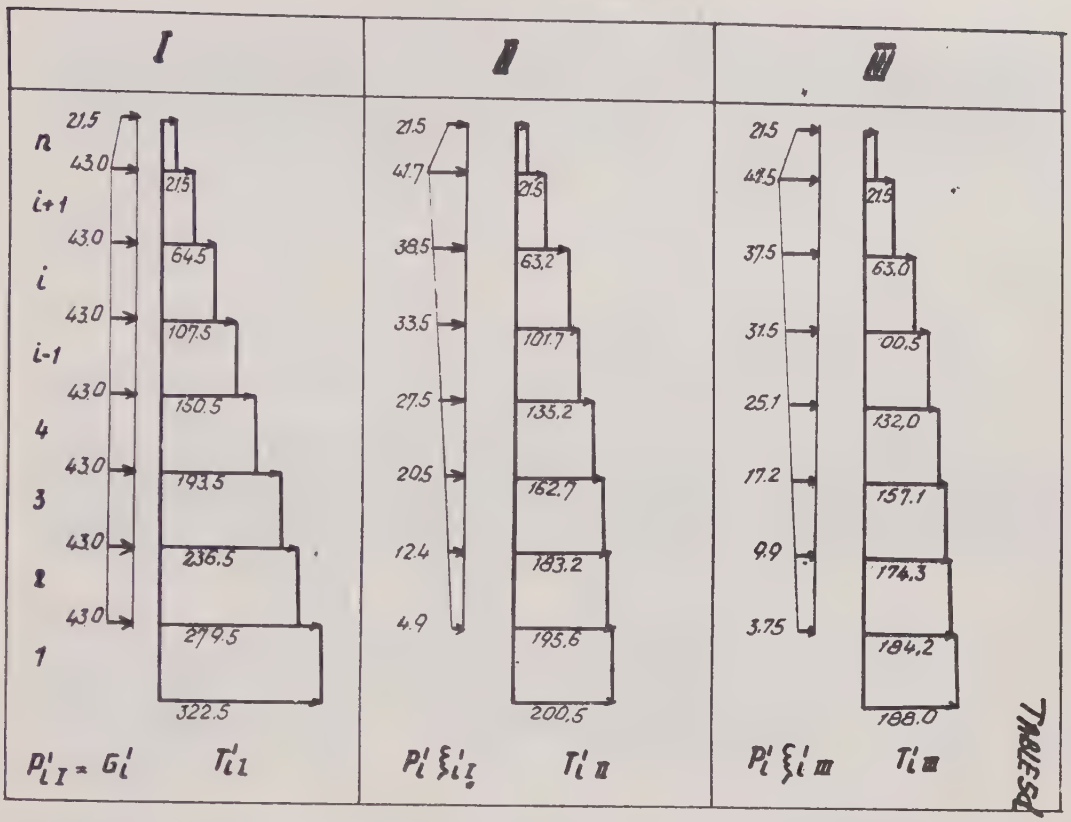
atmospheric factors, under action of gravity and torrential waters.
3. Magnitudes of allowable pressures are only informative.

Table 3

Categories of grounds

Category	Stratification and thickness of the various types of formations		
I Very hard			
II hard			
III firm			
IV soft			
V very soft			

Notes. 1. The thickness of the surface layer should be measured from the surface of the ground downwards.
2. The surface layer should be considered only if a thickness of minimum 2 m of the layer exists under the foundation level.


I
II
III
n
i+1
i
i-1
4
3
2
1
21.5
43.0
64.5
107.5
150.5
193.5
236.5
279.5
322.5
41.7
38.5
33.5
27.5
20.5
12.4
4.9
63.2
101.7
135.2
162.7
183.2
195.6
200.5
44.5
37.5
31.5
25.1
17.2
9.9
3.75
63.0
100.5
132.0
157.1
174.3
184.2
188.0
TABLE 5A

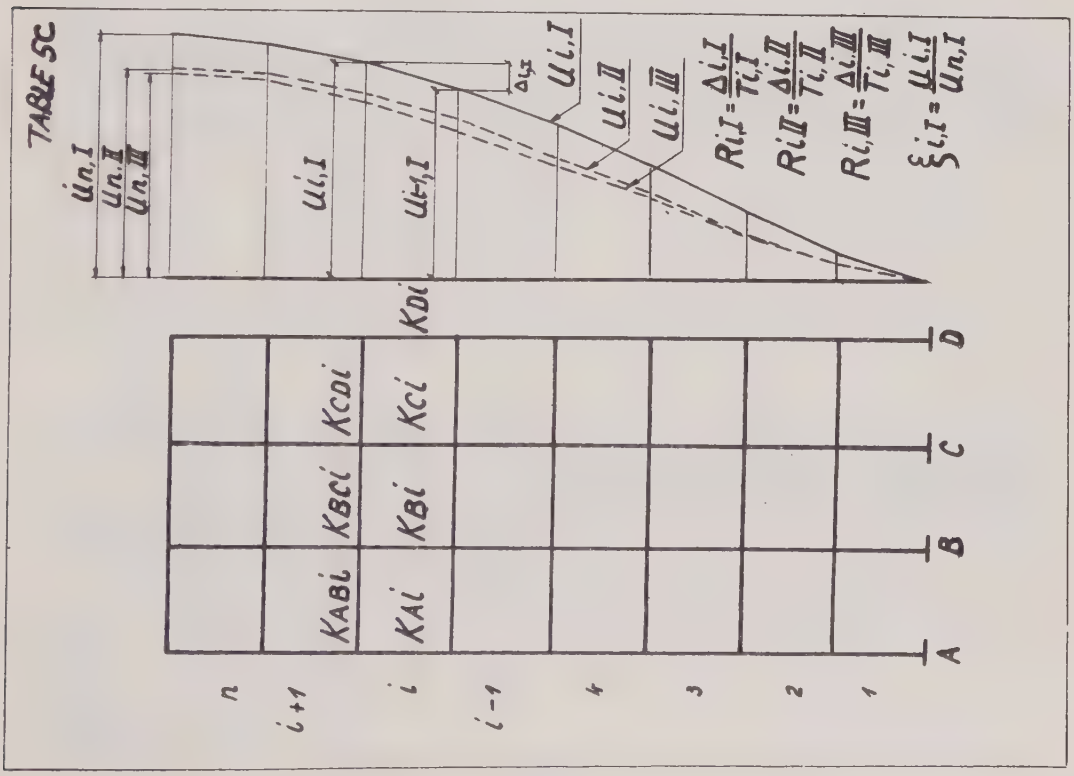


TABLE 5C
Uin,I
Uin,II
Uin,III
Ui,I
Ui-1,I
Δuk
Ui,I
Ui,II
Ui,III
Ri,I = Δi,I / Ti,I
Ri,II = Δi,II / Ti,II
Ri,III = Δi,III / Ti,III
ξi,I = Ui,I / Un,I
Kdi
KCDi
Kci
KBCi
KBi
KABi
KAi
A
B
C
D
n
i+1
i
i-1
4
3
2
1

The Level	h_i	L_i	K_i	K_{i+1}	J_i	β_i	z_i	β_{i-1}	n_i	p_i	z_{i-1}	m_i	n_i
	m	dm3	dm3	dm3	dm3								
	The height of every Level	The same with (44)	The same with (45)	the corresp values of (3)	(3)+(4)	(2)/(5)	2x(6)	The corresp values of (6)	(7)x(8)	The same with (6)	The corresp values of (7)	(10)x(11)	The same with (9)
	1	2	3	4	5	6	7	8	9	10	11	12	13
n													
n-1													
n-2													

The Level	P_i	$T_{i\,sup}$	$E_{i\,inf}$	The approximation	F_i	T_i	h_i	m_i	ε_i	$M_{i\,inf}$	τ_i	$N_{i\,sup}$	M_{i+1}
					tone	tone	m	tm		cm		cm	
	(12)x(13)	(13)/(14)	(12)/(14)		The horiz force applied at the level i	The shear force at the level i	The same with (1)	(18)x(19)	The same with (16)	(20)x(21)	The same with (15)	(20)x(23)	The corresp values of (22)
	14	15	16		17	18	19	20	21	22	23	24	25
n				1									
				2									
				3									
				4									
n-1				1									
				2									
				3									
				4									
n-2				1									
				2									
				3									
				4									

TABLE 5a

The Level	M_i	m_i	K_i	P_i	M_i	L_i	μ_i	μ_{i-1}	P_i	φ_i	h_i	δ_i	Δ_i
	tm	t.m.	dm3	tm/dm3	t.m.	dm3	tm/dm3	tm/dm3	tm/dm3	tm/dm3	m	t.m./dm3	cm.
	(24)x(25)	The same with (20)	The same with (3)	(27)/(28)	The same with (25)	The same with (2)	(31)/(31)	The corresp values of (32)	The same with (29)	(32)+(33)+(34)	The same with (19)	(35)x(36)	(37)/4
	26	27	28	29	30	31	32	33	34	35	36	37	38
n													
n-1													
n-2													

The Level	T_i	R_i	U_i	U_i		L_i	K_i	φ_i	γ_i	COLUMN A	beam AB	COLUMN B	beam BC	COLUMN C	
	tone	tone/cm	cm	cm											
	The same with (28)	The relative level i of (38)	$U_{i-1}+\Delta_i$	The deformation of the engine frame	$K_{Bi}+K_{Ci}$	2x(43)	$K_{Ai}+K_{Bi}+K_{Ci}$								
	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55

TABLE 5b

ASEISMIC DESIGN OF AN OIL REFINERY IN THE STATE OF ASSAM - INDIA

Alexandru Cişmigiu [x] Emilian Tiţaru [xx]

Note:

During 1959 the authors have collaborated in drawing up an aseismic structure for an Oil Refinery, supplied by the Roumanian Government and located in the State of Assam-India. Some specific aspects in the achievement of an aseismic protection are described below:

1. SEISMIC STRUCTURE OF THE STATE OF ASSAM

The State of Assam is known to belong to a zone of earthquakes of considerable strength, situated along the rugged arches of the Himalayas and Burma, being the most active seismic sector of the alpide belt.

The site of the refinery was established near the ridge of the Shillong plateau (near the Brahmaputra river) in the epicentric zone, where the big Indian earthquake of June 12, 1897 occured. This earthquake has been considered as one of the most powerful in history; it was felt over more than 4 million square kilometers and on 75.000 square kilometers all stone constructions were destroyed.

In the epicentric zone the amplitude reached 30 cm. and land settlings reached 12 meters; 5000 aftershocks having occured within the next 10 years.

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2. FORMS OF STRUCTURES TYPICAL TO REFINERIES.

In refineries the following forms of specific structures may be distinguished: distillation columns, oil tanks, horizontal vessels chimney stacks supported by abutments, technological platforms with one or two levels, steam boilers, tubular ovens, thermo-electrical plants, pipe-line systems, sewerage and water supply systems.

The structural and dynamic features of these constructions differ very much, therefore the aseismic solution was examined ffom case to case.

3. CHARACTERISTICS OF THE SOIL-FOUNDATION SYSTEMS

In general the soil may be considered as medium - firm ground. In certain zones lenses of soft-slimy layers may be found; the first thin layers appear at the depth of 4 to 5 meters, and the second one below 15 meters, in thickness up to 10 meters. The following foundation systems have been adopted from case to case: direct foundation or foundations on piles.

4. DETERMINATION OF SEISMIC FORCES

For the determination of seismic forces we received from the Indian refineries the spectrum shown in figure 1. The seismic base shear is determined by eq. (1):

$$Q_B = c.P \qquad (1)$$

in which c is taken from diagram (as shown in figure 1a) The distribution of seismic forces along the height is given by eq. (2):

$$S_i = Q_B \cdot \frac{P_i h_i}{\sum P_i h_i} \qquad (2)$$

Analysis and conformation were carried out in accordance to the Roumanian Specifications (1). The form of this spectrum was identified by the authors to be approximately identical to the medium forms of the acceleration spectra of Californian earthquakes (2). The damping factor "n" has been identified to be about 5% of n_{cr}, corresponding to steel structures. Regarding the use of the this spectrum for structures with several degrees of freedom, it is probable that an equivalence was made for these structures with a one-mass systemusing

an average factor of o,6...o,7. The seismic intensity was considered to correspond to great magnitudes 7-8 at. 5o or more miles from the epicenter.

Compared with Roumanian Specifications the following differences arise: (FIG. 15)

1) The unique force factor "c" does not allow for the differentiation of forces with respect to variations of the damping factor.

2) For the structural forms typical for refineries, the equivalence factor has resulted from calculations to be o,5 - 1,o for the first mode, thus showing important differences as compared with the medium value taken into consideration.

3) Eq. (2) of distribution is valid only for a linear elastic line of mode 1.

4) The influences of higher modes cannot be solved because the corresponding formulas for the equivalence factor and for the distribution of seismic forces, valid for any mode, are missing.

The Roumanian Specification equations, of a general character, entirely allow for the various particularity of the structures of a refinery: (FIG. 15)

$$Q_B = \alpha \cdot \psi \cdot \varepsilon \cdot \mathcal{P} \; ; \quad \psi = \frac{2.92}{\sqrt[4]{n}} \; ; \quad \varepsilon = \frac{[\Sigma P_i \cdot u_i]^2}{[\Sigma P_i][\Sigma P_i \cdot u_i^2]}$$

in which:

α = spectral factor established for $n = o,2o\, n_{cr}$.
ψ = function of damping
ε = equivalence factor
Q_B = base shear

We mention that for extremely rigid structures ($o < T < o,1$sec) with practically no damping, the value of factor C should be increased two or three times.

5. ASEISMIC PROTECTION MEASURES.

5.1. OIL TANKS.

- The volumes of tanks are in the range of 2ooo to 5ooo tons. (FIG. 2)

– The solution of fixed covers was adopted, as this type is less exposed to damage, due to seismic vibrations of the liquid in form of "waves". Measures have been taken to protect the cover against the "seismic wave" acting in an upward motion, by reinforcing the trusses, by providing additional bracings, by fixing the sheet-iron to resist tear and by reinforcing the top outlining ring. The seismic wave action was estimated assuming the liquid surface inclined by 15%. According to the Roumanian Specifications, the distribution of the seismic pressures in a horizontal plane is uneven (see fig.3).

To ensure resistance of the walls against ovalization effects, 2-3 horizontal rigidity rings have been foreseen.

To solve the supporting details of the tank, constructive measures have been taken to prevent the lifting of the sand from below the outlining ring owing to the effect of complex vibrations of the tank, by introducing a concrete belt under it. (See fig.4). Likewise, the basic ring of the tank has been locally strengthened as compared with current solutions (by welding vertical steel corners to it, by thickening the marginal bottom sheet iron, and by introducing a bracket of 100 mm). It is recommended to build the jointing of the tank with the filling and emptying pipes by means of flexible connections.

5.2. DISTILLATION COLUMNS

– The principal types of columns are shown in figure 5.

In determining vibration shapes and periods, besides the flexural deflections of the column, deflections of the supporting frames, as well as rotations due to soil settlements were considered from case to case.

In the case of foundations on piles, the rotation of the foundation was neglected.

Calculus of the periods, forms of vibration and of forces have been made by means of methods developped in paper (1). For this type of structure, the flexibilisation of the lower part (see type 2) due to its support frame and taking into account the rotation of the foundation leads generally to a decrease by 50% of seismic forces.

It is to be mentioned that in a correct analysis (1) when the rotation of the foundation is taken into account, the foundation mass must be included in the force P and in the distribution of Si forces.

It is therefore advisable that foundations should be as light as possible, stable and should permit rotations.

The enchasing of the column with the ground can reduce appreciably the vibration period (up to o,2 sec.) For type (3), where for technological reasons 2 columns of over 500 tons each and two light towers of 55 m. height were grouped on the same foundation mat, each structure was analysed as fixed in the foundation plate.

For light and flexible metal towers, the computed seismic forces have resulted smaller than those produced by wind load.

On the contrary, for columns the bending moment resulting from seismic forces exceeded 2500 tm. Considering working conditions as well as vibration differences, the seismic forces for the design of the foundation plate have been established as follows:

for column Nr.1... 100%, for dolumn nr. 2... 25% and for towers ...50%.

For type 4 we considered that both columns do vibrate in the same phase.

- To ensure resistance, in some cases, the column mantle has been thickened, whereas rigidity rings were provided to prevent buckling.

- For fixing columns into the socle special attention has been given to anchorage bolts. High quality steel was used capable of big plastic strain. We preferred small diameter anchorage bolts places at smaller intervals. It was appreciated that by these measures the dynamic adaptation of anchorages could be increased. It is desirable that constructive solutions be adopted which allow the probable replacement of anchorage bolts, if big plastic strain occurs. Likewise when designing bolts for the creation of strength reserves, an increase of the seismic stress has been allotted to the supporting shoes for fixing bolts and the basic ring has been strengthened accordingly.

5.3. HORIZONTAL VESSELS

The usual types are shown in fig. 6.

Horizontal vessels are characterized in general by a very rigid structure and, in case the foundations are of greater depth, the period can be lowered under o,1 sec. For these structures account must be taken of thermal stress. Maximum forces up to 100% of gravity have been prescribed, acting along any direction in space.

The vessels were tied to the foundations in order to ensure resistance to seismic forces. Fixed supports prevent displacements in every direction. Mobile supports allow the displacement due to thermal variations only alongside the vessel, but prevent the tearing away of the vessel from its supports under earthquake action.

- By tying the foundation legs together by means of strong girders, the longitudinal seismic forces acting on the fixed support are shared by all foundation legs.

- For mobile supports special anchoring devices have been designed.

5.4. PLATFORMS FOR SUSTENSION OF VESSELS

These platforms may have one or more storeys (see fig. 7) and are designed usually as structures in reinforced concrete flexible frames. The analysis of the frames has no special features.

The equivalence factor varies form o,85 to 1,oo. The anchorage of vessels placed on platforms has been designed to resist forces equal to loo% of gravity.

5.5. CHIMNEY STACKS.

- The chimney of the tubular oven is 5o m. high, and is built in steel.

- The influence of vibrations of higher modes was also considered in the determination of seismic forces.

Analysis results have shown that due to reduced mass and to flexibility, the wind stresses were predominant.

5.6. TUBULAR OVEN

In the design of the tubular oven (see fig.8) both seismic forces and thermal effects were taken into account. Transversely, the structure is composed of flexible steel frames. In the longitudinal direction the structure is rigid, because the pillars are braced by means of beams with bolted steel plates.

The roof is braced by trusses and forms a rigid diaphragm.

The rigid gables are separated from the building and are anchored at the roof level with connections in the form of pendulums. This was necessary for two reasons. In the first place to ensure free displacements due to thermal effects. In the second place, the tying of the main structure would have increased the transverse rigidity, consequently also the seismic forces.

As the gables are much more rigid than the frames, this would mean that the seismic forces should have been resisted to only by the gables, which however, would not have enough strength.

At the interior of the steel frame work a thermic insulating layer exists, consisting of refractary brickwork both on the ceiling as well as on the walls.

The refractary brickwork on the ceiling has been suspended by special devices and is lightly braced. The brickwork is anchored to the coating by special joints. The whole refractary brickwork is composed of blocks of special form, each one anchored separately, so that the effect of seismic forces should be resisted to only by the steel frame. Thus the refractary brickwork has been solved as a nonstructural element.

5.7. STEAM BOILER

The construction of the steam boiler is composed of three important parts: the inner steel frame, the refractary brickwork, and the pressure equipment.

For this construction the thermal effects were considered too.

The steel coating which is nonseismic zones is composed only of a simple system of columns and beams, was designed in the case as a system of frames with rigid joints.

To the frame bars the steel suspension plates of the refractary masonry work are attached with bolts.

The upper floor has been solved as a rigid horizontal disk (consisting of sheet iron and beams welded together) so as to ensure spatial cooperation of the system of frames with diaphragms.

With regard to masonry, this has been solved in the same manner as for the tubular oven (5.6.)

The pressurised equipment at the interior of the tank has been fixed to the main construction, so that their thermal expansion should be free, but their rocking be prevented under earthquake action.

5.8. PIPELINE SYSTEM FOR OIL AND REFINING PRODUCTS

The following principal measures were recommended:

1. Weldings should be done according to special working specifications in order to reduce and eliminate initial internal stresses and to prevent diminution of the plastic properties.

2. In general, cast iron fittings should be avoided.

3. Expansion pipes should be provided in a sufficient number in order to avoid buckling and tear by telescopic compression.

4. In the support zones of pipe packages, free displacement should be ensured from case to case; at the same time their rising from the supports should be prevented.

5.9. SEWERAGE AND WATER SUPPLY SYSTEMS

The recommended measures were:

1. The connections with buildings and equipment should be made by means of flexible elements, and the passage through foundations of buildings should be free.

2. For pressure pipes, expansion joints should be foreseen.

3. Connections to concrete pipes should be of semi-flexible type.

4. Distances between visiting chambers should be nearer than in non-seismic zones.

5. Steel pipes should be supported only by very

compact fillings.

5.1o. FIRE PROTECTION MEASURES

Additional measures required in the event of seismic damage causing fire, were considered in addition to normal fire protection measures.

CONCLUSIONS

The experience in designing this Refinery has pointed out that a correct aseismic design should not be limited to the determination of conventional seismic forces isolated from the specific of structural problems and from its mode of construction. A complete aseismic project must be based on the dynamic analysis methods of forces and structures, taking into account all dynamic and constructive properties of typical structures and on a special aseismic conformation followed in every detail.

The aseismic project must take care not only of the design of usual buildings, but it must take into account also the problems of special structures for industrial equipment, for which documentation sources are for the time being very scarce.

It is known that damage of equipment may result in appreciable losses by stopping production.

Although the problems of aseismic design are very complex and scientifically still present many unsolved aspects it is nevertheless possible today to give a complete engineering answer to the practical solution of the most intricate structures and equipment.

N O T E

1. From the technological and aseismic points of view the Refinery project has been entirely worked out by the Design Bureau for Oil Industry in Ploesti Rumania.

2. The measures foreseen for this refinery were

applied also to other structures in the oil industry situated in Roumania or in other countries.

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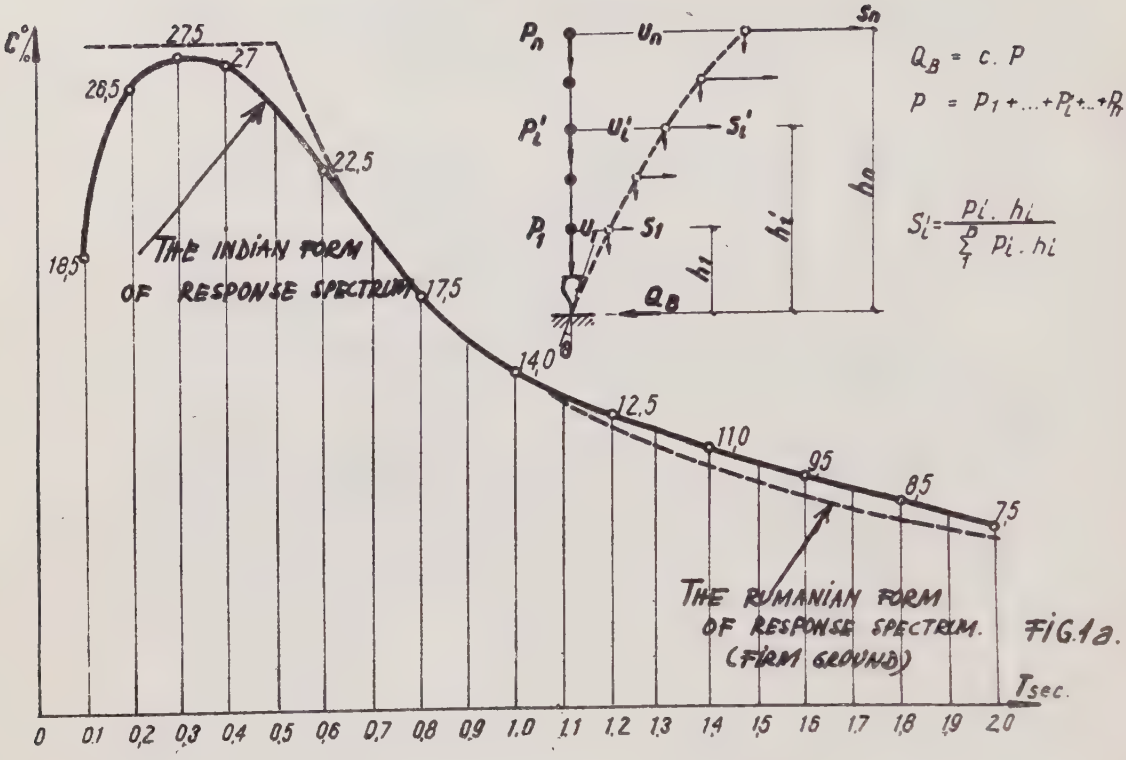


Fig. 1.a. The response spectrum from the Indian Refineries

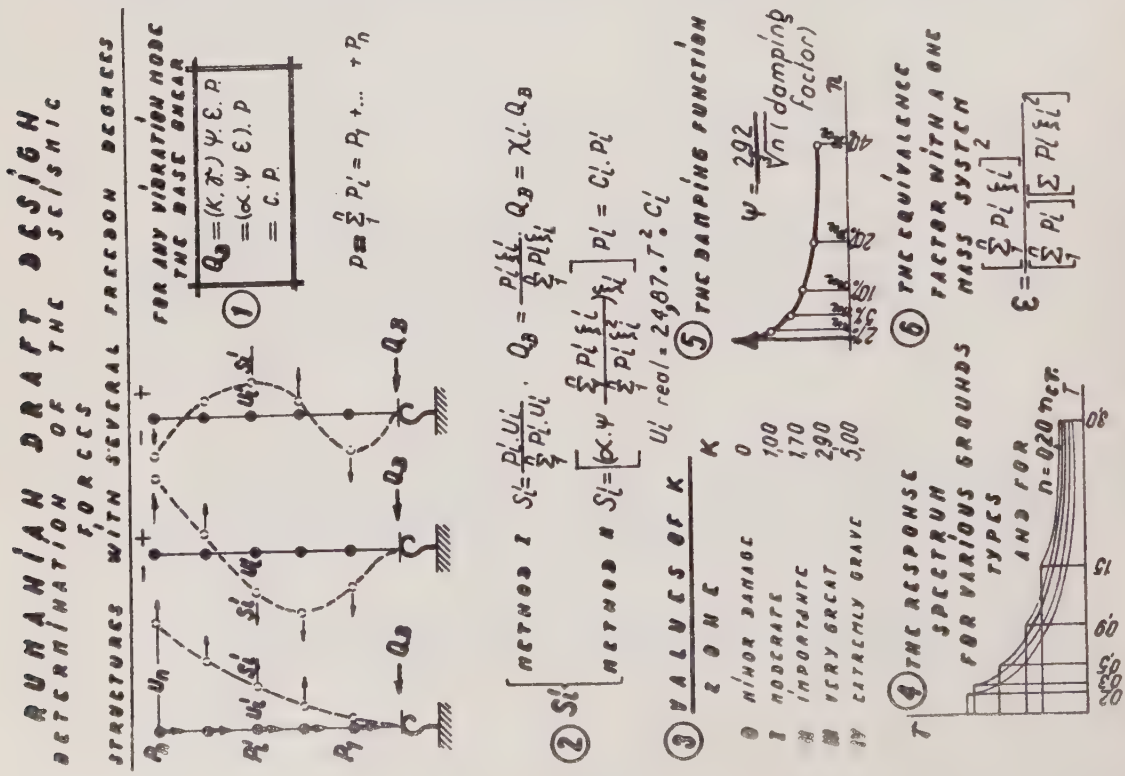

RUMANIAN DRAFT DESIGN
DETERMINATION OF THE SEISMIC FORCES
STRUCTURES WITH SEVERAL FREEDOM DEGREES
FOR ANY VIBRATION MODE THE BASE SHEAR
$Q_B = (K.\beta.\psi.\varepsilon).P = (\alpha.\psi.\varepsilon).P = c.P$
$P = \sum_1^n P_i' = P_1 + \ldots + P_n$
METHOD I $S_i' = \frac{P_i'.u_i'}{\sum_1^n P_i'.u_i'}$ $Q_B = \frac{P_i'\xi_i'}{\sum_1^n P_i'\xi_i'} Q_B = \chi_i'.Q_B$
METHOD II $S_i' = \left[K.\psi \frac{\sum_1^n P_i'\xi_i'}{\sum_1^n P_i'\xi_i'^2}\chi_i'\right] P_i' = C_i'.P_i'$
$u_{i\,real}' = 24{,}87.T^2.C_i'$
VALUES OF K
ZONE K
0 MINOR DAMAGE 0
I MODERATE 1,00
II IMPORTANTE 1,70
III VERY GREAT 2,90
IV CATACLYSM GRAVE 5,00
THE DAMPING FUNCTION
$\psi = \frac{2{,}92}{\sqrt{n\,(\%\,damping\ factor)}}$
n
40
20
10
5
2,5
THE EQUIVALENCE FACTOR WITH A ONE MASS SYSTEM
$\varepsilon = \frac{\left[\sum_1^n P_i'\xi_i'\right]^2}{\left[\sum_1^n P_i'\right]\left[\sum_1^n P_i'\xi_i'^2\right]}$
THE RESPONSE SPECTRUM FOR VARIOUS GROUNDS TYPES AND FOR n=0,20 nCR.
T
3,0
1,5
0,9
0,5
0,2

Fig. 1.b. Rumanian draft design. Determination of the seismic forces

Fig. 2. Oil tanks

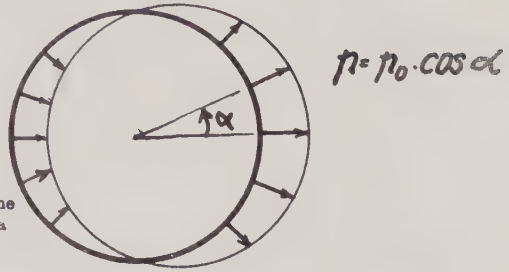


Fig. 3.
The distribution of the seismic pressures in a horizontal plane for tanks.

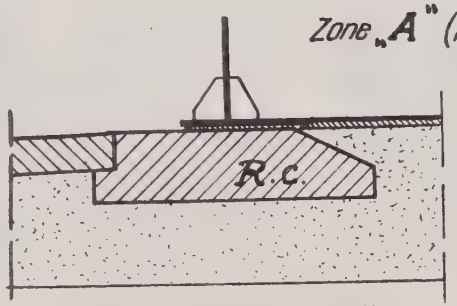


Fig. 4. The base ring. Detail.

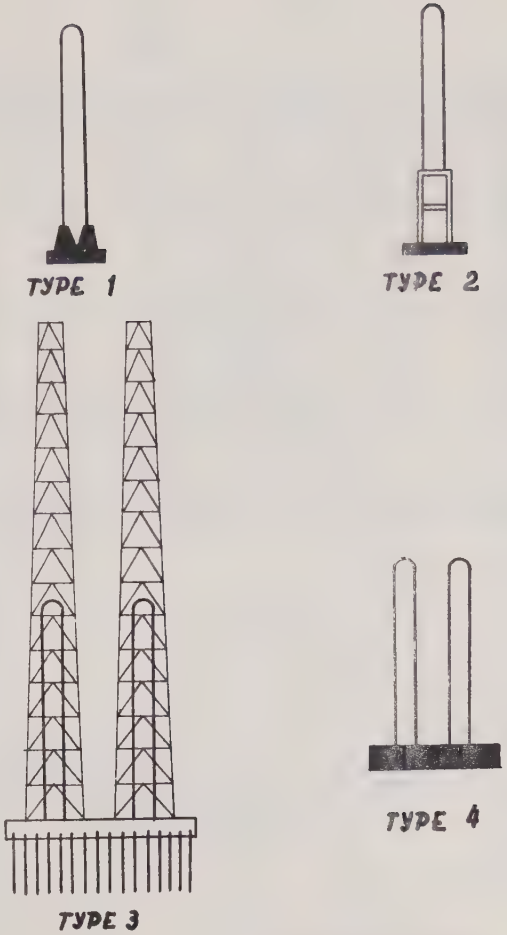


Fig. 5. Distillation columns.

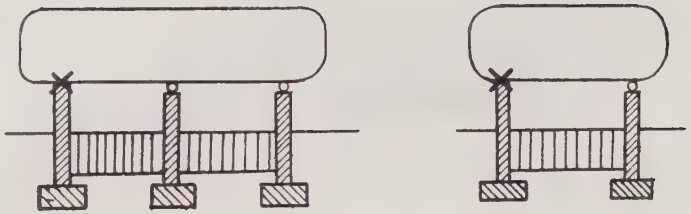


Fig. 6. Horizontal vessels

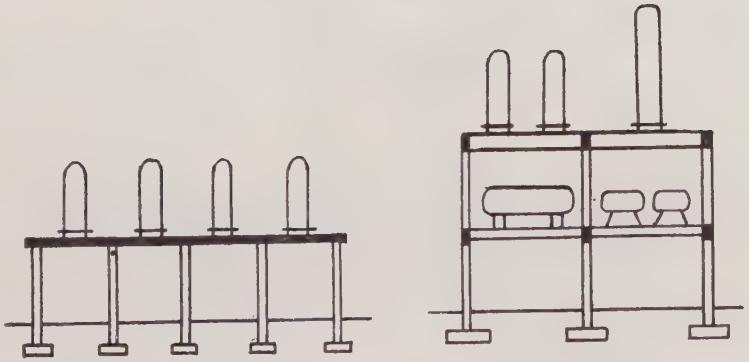
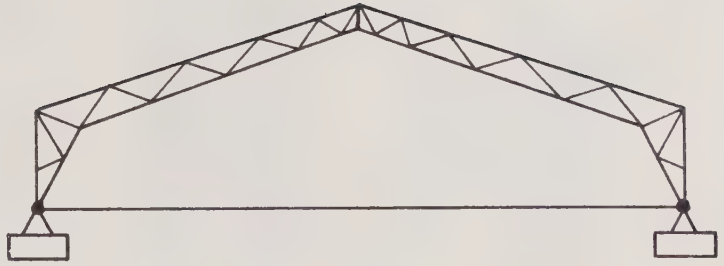


Fig. 7. Platforms for sustension of vessels



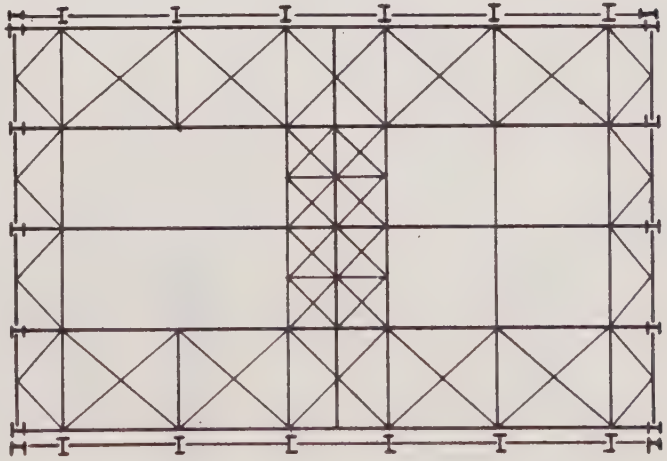


Fig. 8. The tubular oven

ON ASEISMIC RESISTANCE OF STRUCTURES TAKING INTO ACCOUNT THE DIFFERENT BEHAVIOUR OF MATERIALS TO SHOCKLOADS, WITH SPECIAL APPLICATION TO THE ASEISMIC FORM OF SHELLS

by

Nicolae Korcinski*

Earthquake action on a structure is made manifest by the impact of an external energy. The impact may be applied as:

a. - A seismic shock (fig. 1 and 2), if the building is situated not too far away (in a terrestrial scale) from the epicentre.

This is the case of the city of Bucharest situated at 170 km from the focus of earthquakes in Rumania.

b. - An accelerated vibration movement (fig. 3) when the building is far away from the epicentre.

This distinction is considered to be important, as the material of the structure may have a quite different behaviour to the two categories of impacts mentioned above.

In the first case the structure recives the impact of a sudden seismic shock, whereas in the second the building gets into a pondular movement due to the oscillation of the soil of an increasing amplitude.

The yield point of the constructional steel is fairly increased when a shock stress is applied (fig. 4). Constructional steel is so far a kind of a "quick" material, quickly adapting itself in the elastic range to sudden loads. Therefore steel structures subject to an earthquake stand fairly well. As the increase in stresses due to earthquake (up to a fraction of the gravitational value) does not attain the factor of increase of the yield point, it may be set forth that steel structures subject to seismic shocks do not need any special measures regarding their elastic resistance.

Measures have to be taken through regarding the stability according to the axial forces increased. To increased vertical loads the horizontal seismic loads are added, giving birth to the transverse buckling. However the influence of transverse loads on the critical buckling load is relatively small (fig. 5); thus the influence of this horizontal load on the stability of structure may be neglected (fig. 6 and 7).

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Regarding the behaviour of reinforced concrete structures, the phenomenon as above described, of the different behaviour of a material under shockloads, as compared with that under static loads is not well elucidated.

Researches are carried out at the Chair for Reinforced Concrete in the Bucharest Institute for Civil Engineering by Prof. V.Nicolau. The first results of this researches, as well as remarks made directly on reinforced concrete structures damaged by the 1940 earthquake of the VIII-th degree in Bucharest (the photographs delivered by courtesy of Prof. A.A.Beleș), alow us to set forth some conclusions:

Concrete itself is a "lazy" material which does not develop rapid elastic strain when acted on by sudden loads, and so far by reinforced concrete structures the shock load increment is transmitted to the rods. This causes rods to be subject to a greater strain than the concrete itself, and to part from the latter.

Damaged reinforced concrete struts present the buckling of rods and parted from the concrete. As the rods in the struts are rather understressed by static loads it was to be presumed that this state of stress would remain under the increased axial load due to earthquke. The strong buckling of the rods and a small damages in the concrete show that the rods were given the main part of the load increment. Struts should be therefore checked to the axial load increased the entire load increment alloted to the rods, with checking the buckling of rods between stirrups (fig. 9).

The direction and width of cracks in beams, their occuring near the joints, show that the oblique (even surface) overstrained bars parted from the concrete, when no important cracks were observed at the bays of the beams. The beams were thus damaged as a consequence of shear force increase than by the bending moment increase (fig. 10).

The concrete being not capable to resist a sudden shear force increase, the latter has to be entirely alloted to the bars. The same as to the horizontal earthquake shear loads acting on the struts, which also have to be entirely alloted to bars, even to oblique bars especially provided for at the base of struts. The sections of struts should not be too narrow.

When a seismic shock occurs plastic strain in the concrete can't be taken into account and the whole load increment has to be given to the bars which if possible these should not be even (in order to encrease steel-concrete adhesiveness). Plastic structural analysis can and should be applied to structures situated for away (in a terrestrial scale) from the epicentre, where the earthquake is manifest as a wave of an increasing amplitude and the concrete "has his time" to follow the strains of the bars.

* The damaged struts provided with a supplementary rods, were bandaged and fresh concrete torkreted; they behave fairly well in subsequent earthquake of the IV-th degree (fig. 11)

The energy alloted to a shell by an earthquake is stored in two forms (fig. 12 and 13):

a. - one due to stresses with a resultant in the medium surface of the shell, denoted by U_o and

b. - another due to the elastic bending of the shell denoted by U_i.

In order to resist an earthquake, the shell must be capable to absorb as much energy as possible. The energy stored in the form U_o is greater than that in the form U_i, as in the first case the whole section of the shell works up to the strength limit, whereas in the second, the energy up to the strength limit can be stored only by the extreme fibres on both faces of the shell (seismic energy applied as a shock, as previously mentioned).

To the more, in regions where the shell works as a sail it stores energy almost exclusively in the U_o form. Only in sections where the influence of the function of displacements (w) is noticeable it stores energy in the U_i form unfavourable for the purpose.

Thus, in order to store maximum energy a shell (for example a cupola) most be high enough, as in such a case the influence of (w) is limited to a region in the vicinity of supports, the rest of the shell working as a sail.

The energy stored in the region of the shell working as a sail is:

$$U_o = \frac{h}{2E}\iint\left[\sigma_x^2 + \sigma_y^2 - 2\mu\sigma_x\sigma_y + 2(1+\mu)\tau^2\right] dx\,dy$$

Referred to the principal axes at the considered point, of the shell, the expression to be integrated has the form:

$$\sigma_1^2 + \sigma_2^2 - 2\mu\sigma_1\sigma_2$$

Poission's factor (μ) having a subunit value (for instance for reinforced concrete $\mu = 1/6$) the expression to be integrated, and consequently the energy stored in the shell, shall reach a maximum value when ($\sigma_1 = \sigma_2 = \sigma_o$) that is when the shell has a membrane form. In this case the unit stresses (σ_o) in the medium surface shall vary in the same proportion as the external load. In the region of a sail state, the shell being statically determined the load variation gives birth to an increment of the stress (σ_o) and not to a change in form worth to be taken into account. It will be enough to consider a safety factor equal to that increasing the vertical load. With these increased stresses the shell should be checked on buckling !

To achieve a membrane or a sail state a shell needs simple supports along their borders, as the reaction should be in the plane tangent to the medium surface (fig. 14). This can be achieved by placing the shell on rolls or spheres (in the case of cupolas). Though this leads

to construction difficulties the realization of such supports for cupolas of a not too great span are not insurmontable (for example the mobile cupolas of astronomical observatories.

In the case of big span shells the supports give birth to bending moments in the shell due to the influence of the displacement function (w) - known as the influence of supports !

For high and elevated shells the influence of the supports is noticeable only over a relatively small distance (fig. 15 and 16), and the damages on reinforced concrete cupolas of Bucharest by 1940 earthquake confirm this behaviour of high shells (Photo 17).

For reasons given in the previous paper, the seismic load increment should be allotted to the bars, in zones where the influence of supports is of importance. In such zones, double reinforcement must be laid.

In samll-hight shells the influence of supports is felt over a large part if not over the entire shell. Such shells work predominatly on bending, being consequently unsuitable for the purpose, even if they have an initial form of a membrane according to the given external load. The variations (κ_x, κ_y) of curvatures, which are of importance in such a case result in absorbtion by the shell of the energy

$$U_i = \frac{D}{2}\iint[(\kappa_x+\kappa_y)^2-2(1-\mu)(\kappa_x\kappa_y-\chi^2)]\,dx\,dy$$

smaller than that which could be absorbed in U_o form.

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Fig. 1

Fig. 2

Fig. 3

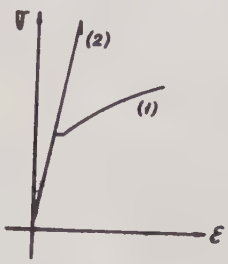
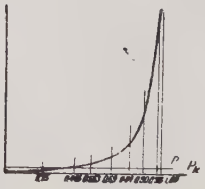


Fig. 4

Fig. 5

Fig. 6



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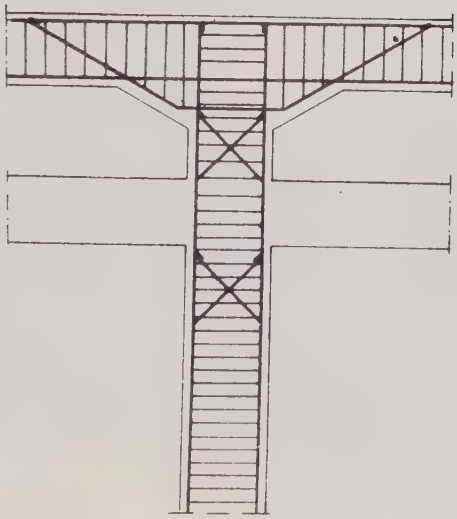
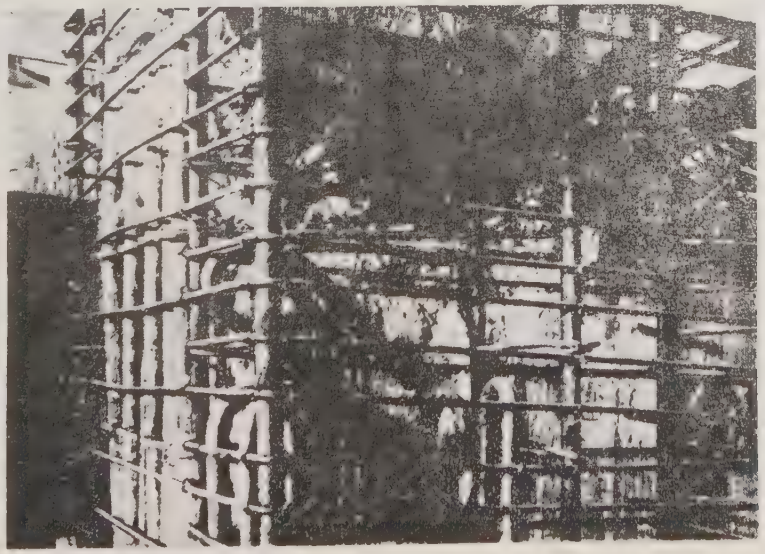


Fig. 8

Fig. 9

Fig. 10



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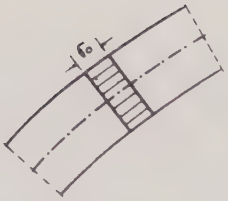

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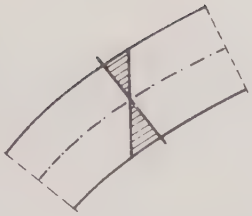

Fig. 13

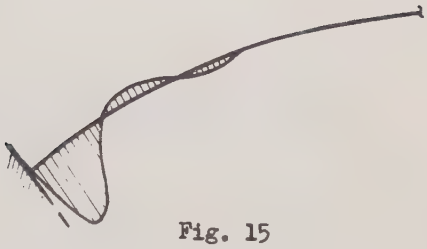

Fig. 15

Fig. 14

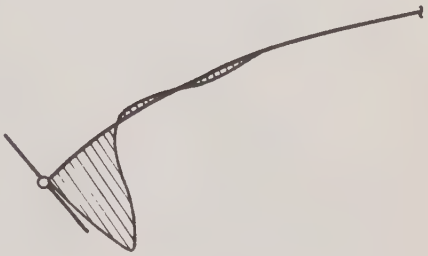

Fig. 16

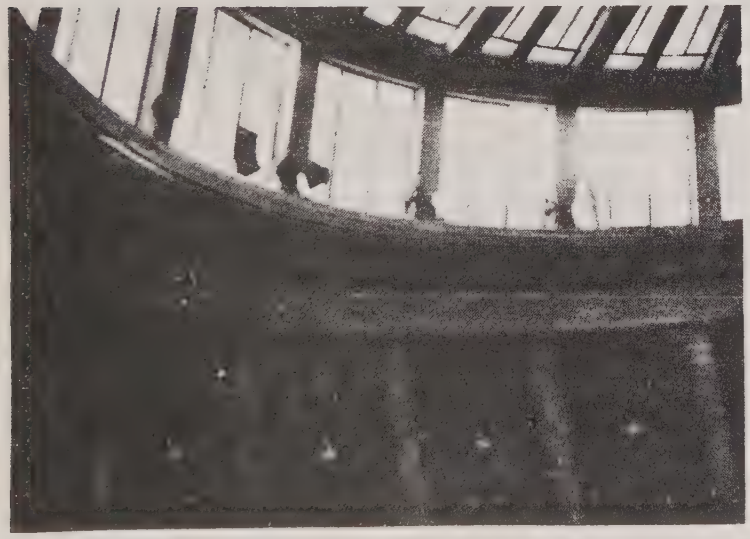

Fig. 17

Session Summary

by

Toshihiko Hisada*

In the field covering Earthquake Resistant Design, Constructions and Regulations, 32 papers were presented to the Conference. Due to time limitation, 21 of them were presented oralley at the session, while the rest were reported in summarized form.

As to the Earthquake Resistant Regulations, there were 7 presentations from the countries located in seismic zones in the world. Mr. M. Numata reported on the present Japanese regulations concerning civil engineering structures, including dams, bridges and harbor structures, Messrs. W. Binder and W.T. Wheeler on the Proposed Building Code Provisions (1959) of Structural Engineers Association of California, U.S.A., Mr. E. Rosenblueth on the Aseismic Provisions (1960) of the Federal District in Mexico, Messrs. V.A. Bikhovsky, J.I. Goldenblat and J.L. Korchynski on the Aseismic Standards and Code (1957) of U.S.S.R., Mr. J.F. Borges on the Portuguese Code (1958), Messrs. Em. Titaru and Al. Cismigiu on the Rumanian General Design Specifications for Civil and Industrial Buildings in seismic area and Mr. A. Brenier on French Regulations for Civil Engineering Structures in Seismic Regions.

Among those presentations, the regulations of U.S.A., U.S.S.R., Mexico and Rumania adopt the dynamic methods, considering the acceleration spectra in connection with the natural periods of buildings and civil structures, and the Portuguese Code employs the method of limit analysis. These are considered to reflect the latest trends in the earthquake resistant design.

There were 10 reports embodying personal proposals or opinions on the earthquake resistant designing. Those include the papers by Mr. S.V. Medvedev on dynamic analysis based on the earthquake records in U.S.S.R., Mr. Y. Otsuki on the dynamic design method of reinforced concrete structures, Mr. E. Rinne on the overturning moment of tall buildings in connection with the SEAOC Code, Mr. S. Tani on the arrangement of earthquake resistant members in the buildings, Messrs. J. Krishna and S. Menayas on dynamic design and Mr. R.N. Joshi on the seismo-free construction. In regard to earthquake resistant designing of dams, Mr. N.N. Ambraseys, of Great Britain, and Messrs. M. Hatanaka and I. Minami presented their respective original views.

As regards the earthquake resistant design of various constructions, there were presented 8 papers which included the following: Messrs. Y. Tsuboi and M. Kawaguchi on flat slab and shell constructions, Mr. N. Korchinsky on reinforced concrete construction, especially on shell structure, Mr. S. Sachanski on reinforced concrete frames having infilling masonry walls, Mr. K.S. Zabriev on precast construction employing large concrete blocks or panels now in use in U.S.S.R., Messrs. T. Naka, M. Wakabayashi and S. Takada on steel-reinforced

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concrete construction which is commonly used for high-story buildings in Japan, and Messrs. K. Takeyama, T. Hisada and Y. Ohsaki on wooden construction in Japan. Mr. H.J. Degenkolb reported the earthquake resistant design of small buildings in U.S.A., while Messrs. J.G. Bouwkamp and J.F. Meehan presented the results of their experiments on the drift limitation imposed by glass in window sash.

Four papers introducing examples of earthquake resistant design included the following reports on actual designs: Mr. A. Zeevaert on the 43 storied Tower Latino Americana in Mexico City which became famous by escaping any damage by the earthquake of 1957, Messrs. D. Vergun and D.V. Whitmore on two long-span multistory buildings in U.S.A., Messrs. R. Hicks and I.A.B. Gaunt on the nuclear power station which is now under construction at Tokai Village in Japan, and Messrs. Em. Titaru and Al. Cismigiu on an oil refinery in the State of Assam in India.

Two pairs of Japanese researchers, Messrs. S. Ban and H. Muguruma and Messrs. T. Hatano and H. Tsutsumi, submitted their respective reports on the dynamical strength and rigidity of concrete when subjected to rapid loading such as the shock of an earthquake. As a general report, Messrs. A.A. Beles and M. Ifrim presented a paper on the recent development of earthquake engineering in Rumania.

A remarkable trend reflected in many reports submitted in Session IV is that dynamic design method based on the elastic response spectra obtained by actual seismographic records in gradually coming to be adopted in many seismic countries in the world.

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